

1990

Analysis and design for frost effect in soil-steel structures.

Nabil. Ghobrial
University of Windsor

Follow this and additional works at: <http://scholar.uwindsor.ca/etd>

Recommended Citation

Ghobrial, Nabil., "Analysis and design for frost effect in soil-steel structures." (1990). *Electronic Theses and Dissertations*. Paper 1154.

This online database contains the full-text of PhD dissertations and Masters' theses of University of Windsor students from 1954 forward. These documents are made available for personal study and research purposes only, in accordance with the Canadian Copyright Act and the Creative Commons license—CC BY-NC-ND (Attribution, Non-Commercial, No Derivative Works). Under this license, works must always be attributed to the copyright holder (original author), cannot be used for any commercial purposes, and may not be altered. Any other use would require the permission of the copyright holder. Students may inquire about withdrawing their dissertation and/or thesis from this database. For additional inquiries, please contact the repository administrator via email (scholarship@uwindsor.ca) or by telephone at 519-253-3000ext. 3208.



National Library
of Canada

Bibliothèque nationale
du Canada

Canadian Theses Service

Service des thèses canadiennes

Ottawa, Canada
K1A 0N4

NOTICE

The quality of this microform is heavily dependent upon the quality of the original thesis submitted for microfilming. Every effort has been made to ensure the highest quality of reproduction possible.

If pages are missing, contact the university which granted the degree.

Some pages may have indistinct print especially if the original pages were typed with a poor typewriter ribbon or if the university sent us an inferior photocopy.

Reproduction in full or in part of this microform is governed by the Canadian Copyright Act, R.S.C. 1970, c. C-30, and subsequent amendments.

AVIS

La qualité de cette microforme dépend grandement de la qualité de la thèse soumise au microfilmage. Nous avons tout fait pour assurer une qualité supérieure de reproduction.

S'il manque des pages, veuillez communiquer avec l'université qui a conféré le grade.

La qualité d'impression de certaines pages peut laisser à désirer, surtout si les pages originales ont été dactylographiées à l'aide d'un ruban usé ou si l'université nous a fait parvenir une photocopie de qualité inférieure.

La reproduction, même partielle, de cette microforme est soumise à la Loi canadienne sur le droit d'auteur, SRC 1970, c. C-30, et ses amendements subséquents.

**ANALYSIS AND DESIGN FOR
FROST EFFECT IN
SOIL-STEEL STRUCTURES**

by

NABIL GHOBRIAL

A Thesis
Submitted to the
Faculty of Graduate Studies and Research
through the Department of
Civil and Environmental Engineering
in Partial Fulfillment of the Requirements
for the Degree of Master of Applied Science at
The University of Windsor

Windsor, Ontario, Canada
October, 1990



National Library
of Canada

Bibliothèque nationale
du Canada

Canadian Theses Service Service des thèses canadiennes

Ottawa, Canada
K1A 0N4

The author has granted an irrevocable non-exclusive licence allowing the National Library of Canada to reproduce, loan, distribute or sell copies of his/her thesis by any means and in any form or format, making this thesis available to interested persons.

The author retains ownership of the copyright in his/her thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without his/her permission.

L'auteur a accordé une licence irrévocable et non exclusive permettant à la Bibliothèque nationale du Canada de reproduire, prêter, distribuer ou vendre des copies de sa thèse de quelque manière et sous quelque forme que ce soit pour mettre des exemplaires de cette thèse à la disposition des personnes intéressées.

L'auteur conserve la propriété du droit d'auteur qui protège sa thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

ISBN 0-315-65167-9

© NABIL GHOBRIAL, 1990

I hereby declare that I am the sole author of this document.

I authorize the University of Windsor to lend this document to other institutions or individuals for the purpose of scholarly research.

NABIL GHOBRIAL

I further authorize the University of Windsor to reproduce this document by photocopying or by other means, in total or in part, at the request of other institutions or individuals for the purpose of scholarly research.

NABIL GHOBRIAL

To My Parents.

ABSTRACT

Underground conduits are being built in increasing numbers and size for use as bridges, underpasses and tunnels. The main incentives of using soil-steel structures are the low costs and high productivity. A recent survey by the Canadian Society for Civil Engineering (CSCE) Committee on soil-steel structures revealed that a relatively large number of conduits are in an apparent state of distress. In the absence of adequate information on the possible reasons of distress, the effect of soil freezing is studied in this thesis as one of the possibilities of the distress.

The author believes that the freezing and thawing of the soil around the culvert is one of the reasons for the distress in soil-steel structures especially in case of pipe arches. Therefore, the effect of the frost action on roads and highways is clearly explained herein and presented as a complete literature survey to which the designers of the soil-steel structures can refer. This study is then applied to analyze the effect of this phenomenon on soil-steel structures and their durability. Also, an experimental model was designed to simulate the loss of soil support around the pipe arch and thus observe and study its effect on the structure. Additionally, an evaluation to the present codes is made from the standpoint of frost action.

Through the understanding of the problem of frost action and the theory of soil-steel structure, the frost action problem can be controlled. A few recommendations are presented in this study to avoid the frost action and to overcome the distress. Also, cement-modified soil is proposed to improve the properties of the available soil in the site instead of replacing it with naturally non-frost susceptible soil.

ACKNOWLEDGEMENTS

The level of appreciation and respect that I feel towards my advisor, Dr. G.A. Sayed, for his important suggestions and continuous encouragement cannot be expressed in written form, but it will be remembered for the rest of my life. To him I say "Thank you."

The author would like to thank Dr. J.T. Laba, for his guidance throughout this work.

Thanks are due to Mr. G. Kuhun, Mr. F. Kiss and the Central Research Shop staff members for their help during the experimental investigation.

The financial assistance given by the Ontario Ministry of Transportation is sincerely appreciated.

Special thanks are extended to Mrs. N. Eissa for drafting the figures.

Finally, the author is grateful to his family for their support and encouragement.

TABLE OF CONTENTS

ABSTRACT	v
ACKNOWLEDGEMENTS	vi
Chapter I: INTRODUCTION	1
Soil-Steel Structures	1
Observed Distresses in Soil-Steel Structures	2
Frost Action on Soil	3
Objective	4
Chapter II: LITERATURE SURVEY AND FREEZING THEORY	10
Climate	10
Frost Heaving Mechanism	11
Structure of Frozen Ground	12
Stratified Frozen Soil	12
Massive or Homogeneous Frozen Soil	13
Process of Soil Freezing	13
Ice Crystallization	13
Moisture Migration	14
Factors Affecting The Frost Heave	15
Grain Size	16
Rate of Freezing	16
Depth of Frost Penetration	17
Applied Pressure	17
Compaction	17
Degree of Saturation	18
Thawing and Refreezing	18
Criteria of Frost-Susceptibility	19
Beskow's Criterion	19
Casagrande's Criterion	20
Heaving Pressure	20
Reasons for The Heaving Pressure	21
Heaving Pressure Prediction	23
Everett and Haynes (1965)	24
Penner (1966)	25
Williams (1966)	25
Application of Heaving Pressure Theory	27

Application of Heaving Pressure Theory on Casagrande's Criterion	28
Water Expulsion During Freezing	29
Frost Boil	30
 Chapter III: FREEZING EFFECT TO SOIL-STEEL STRUCTURES	43
Application of The Soil Freezing Theory on Soil-Steel Structures	43
Thermal Analysis	43
During Freezing	44
During Thawing	44
Structural Analysis	45
Numerical Example	45
Laboratory Model	47
Soil Container	47
Soil	47
Culvert	47
Wooden Dowels	48
Hydraulic Ram	48
Instrumentation	48
Compaction Procedures	49
Experimental Program	49
Test Results	50
 Chapter IV: TREATMENT	73
General	73
Remedies For Frost Action	74
Chemical Additives	74
Thermal Insulation	74
Portland Cement	76
Cement-Treated Soil	77
Cement-Modified Soil	78
Frost Action Effect On Cement-Modified Soil	79
Soil Samples	80
Experimental Program	81
Grain Size Analysis Test	81
Frost Action Test	82
Description of Equipment	82
Sample Preparation	83
Samples without cement	83
Samples with Cement	84
Test Procedures	85
Test Results	85

Chapter V: EVALUATION AND RECOMMENDATION	100
The Structure of Backfill Material	100
Soil Envelope Dimension	102
Soil Material of The Embankment	103
Construction Technique	103
CONCLUSIONS	104
 Appendix A: PIPE ARCH EXPERIMENT MODEL	 106
Summary of soil properties	106
Mechanical properties of the culvert material	106
 Appendix B: EQUIPMENT SPECIFICATIONS	 107
Pipe Heating Cable	107
Thermistor Probe	107
 REFERENCES	 108

LIST OF TABLES

2.1	Beskow's Criterion of Frost Susceptible Soils.	42
4.1	Bearing Value of Cement-Modified Granular Soil.	97
4.2	Geotechnical Properties of Soils A, B, and C.	98
4.3	Summary of The Frost Heaving Test Results on Soils A, B, and C.	99

LIST OF FIGURES

1.1	Soil-Steel Structures During Various Stages of Construction	5
1.2	Theoretical Interface Radial Pressures Around Various Shapes of Soil-Steel Structures.	6
1.3	Crimps at The Haunch of a Pipe-Arch Structure.	7
1.4	Joint Rusting in a Pipe-Arch Structure.	8
1.5	Bolt Tearing at The Top of The Corner Plate of a Pipe-Arch Structure.	9
2.1	Relation Between Freezing Index and Depth of Frost Penetration.	33
2.2	Freezing Index Map in Canada.	34
2.3	Freezing of Soil Moisture.	35
2.4	The Boundary Between Growing Ice-Crystal and The Underlying Soil Particles.	36
2.5	The Relationship Between Frost Heaving and The Rate of Freezing.	37
2.6	The Effect of Applying Pressure on Frost Heave.	38
2.7	Freezing of Non-Saturated Soil	39
2.8	Everette and Hayness Particles Model.	40
2.9	Relation Between Experiment and Theoretical Heaving Pressure.	41
3.1	Temperature Profile of The Soil-Culvert Interaction.	52
3.2	Effect of Heaving Pressure on Pipe-Arch Conduit (Numerical Example)	53
3.3	Corrugation Profile of The Conduit Used in The Laboratory Model.	54
3.4	Inside Dimension of The Culvert Used in The Laboratory Model.	55
3.5	Detailed Drawing of The Experimental Model	56
3.6	Strain and Dial Gauge Locations.	57
3.7	The Conduit Before Placement in The Container	58
3.8	The Experimental Set-Up Before The Backfilling Operation.	59

3.9	The Bedding Shape Before Placing The Conduit.	60
3.10	Placement of The Wooden Dowels Through The Soil.	61
3.11	The Experimental Set-Up Before Applying The Loads.	62
3.12	Beginning of Crown Deflection.	63
3.13	Formation of Plastic Hinges at Crown and Shoulders.	64
3.14	The Conduit Shape at a Load of 8 kN After Pulling All The Dowels. . . .	65
3.15	The Conduit Shape After Unloading.	66
3.16	The Bending Moment Diagram Before Pulling The Dowels.	67
3.17	The Bending Moment Diagram After Pulling The Dowels.	68
3.18	The Axial Force Diagram Before Pulling The Dowels.	69
3.19	The Axial Force Diagram After Pulling The Dowels.	70
3.20	The Deflection Before Pulling The Dowels.	71
3.21	The Deflection After Pulling The Dowels.	72
4.1	Grain Size Distribution for Soils A, B, and C Without Cement.	87
4.2	Grain Size Distribution of Soil A With Cement.	88
4.3	Grain Size Distribution of Soil B With Cement.	89
4.4	Grain Size Distribution of Soil C With Cement.	90
4.5	Detailed Drawing of The Frost Cell.	91
4.6	Cross-Section of The Frost Cell.	92
4.7	The Frost Cell Inside The Freezer.	93
4.8	Experimental Heave of Different Percentages of Cement With Soil A.	94
4.9	Experimental Heave of Different Percentages of Cement With Soil B. . .	95
4.10	Experimental Heave of Different Percentages of Cement With Soil C. . .	96
5.1	Freezing of Sand Containing a Thin Layer of Silt.	105

Chapter I

INTRODUCTION

1.1 Soil-Steel Structures

Soil-steel structure is the name given to bridge, culvert, and underpass type of conduit opening designed with corrugated curved steel plates surrounded by compacted soil. In the last 30 years many obsolete drainage structures in Canada and United States have been replaced with soil-steel structures. These structures, which have spans up to 18 m, are very popular because they are relatively easy to construct and are considerably cheaper than their conventional counterparts, viz., concrete box culverts and short-span slab bridges. The average savings is 30% over short span bridge alternatives.

Soil-steel structures are composed of mild carbon steel, galvanized and corrugated plates. These plates can be rolled to different radii so that they form various cross sections such as circular, elliptic, or pipe arch. The sheets are punched along the edges so that they can be bolted together. The construction of the soil-steel structures begins with bolting the sheets together to form a flexible steel conduit. At the same time, the bedding is pre-shaped to the invert radius of the conduit without compaction to allow structure relaxation into bedding. The backfill material is then placed and compacted in layers on each side of the conduit. Each layer is compacted to a minimum 85% standard proctor density. Compaction close to the wall, 0.5 to 0.75 m, is done with a hand held compactor. Then the embankment materials are placed and compacted over top of the soil steel structure. Fig. 1.1 shows the sequence of the backfilling operation and the

resultant changes in the cross section due to compaction. Also, it shows the soil pressure on the conduit.

The theory of soil-steel structure assumes that after completion of construction the flexible steel conduit develops composite action with the surrounding soil. Because the conduit wall itself has relatively little inherent strength to sustain the loads on it, a large part of its ability to support loads is due to the development of lateral soil pressure. As a result of the high degree of flexibility of the conduit wall, the loads imposed on the pipe are resisted mainly by the membrane action of the wall. This phenomenon implies that the radial soil pressure on the conduit wall is approximately inversely proportional to the local radius of curvature of the conduit as illustrated in Fig. 1.2. In case of pipe arches which were found to be very susceptible to distress [47], the soil pressure will be as shown in Fig. 1.2.C which shows that the lowest soil pressure is on the bottom plate while the greatest pressure is on the corner plate. Therefore, a very stable and strong soil is required outside the corner plate to support this high corner pressure.

1.2 Observed Distresses in Soil-Steel Structures

Many soil-steel structures were observed to be in distress especially in Ontario and a few of the northern American States [47]. The Ministry of Transportation of Ontario (MTO) with the help of some other structural experts have investigated some of these structures. An intensive inspection of distressed structures was conducted in the Elgin County and adjacent counties of Southwestern Ontario in 1985. The main purpose of the inspection was to categorize the various types and stages of distress. Among 141 soil-steel structures inspected, 85 were pipe arches and of those 56 showed some signs of distress. The most prevalent type of distress in pipe arch structures was the presence of

horizontal cracks running through the valley bolts at the top of the corner plate. In some cases these cracks met each other across the top of the ridge corrugation. Furthermore, uplift was observed at the bottom plates as well as deflection in the top plate of pipe arch structures which caused the horizontal dimension to increase. However, a decrease in the horizontal dimension and increase in the vertical dimension were also reported in some cases during the winter season. Also, it was noticed that the frequency of distress was associated with the absence of head walls.

In Sept. 1989, a team from the University of Windsor including the author visited three sites of pipe arch structures in distress. The following have been observed:

1. The existence of crimps at the haunch along the whole length of the conduit as shown in Fig. 1.3.
2. Distortion of some culverts which probably took place during construction.
3. Excessive rusting, especially in the joints where it could have started at the bolt holes as shown in Fig. 1.4.
4. Bolt tearing was noticed to become excessive near the middle of the culvert as shown in Fig. 1.5.
5. Deflection in the top of the conduit.
6. Upheaval in the floor of the conduit.

1.3 Frost Action on Soil

It is believed that most of the observed distress in pipe arch structures has occurred through a loss of soil support around the structure particularly at the corner plate where the highest pressure is concentrated. Thus the durability of soil-steel structures is governed not only by the culvert material but also by the stability of the surrounding soil.

This soil must be strong and stable even through different climatic conditions such as freezing and thawing. The most troubling effect of soil freezing is the phenomenon of "Earth Heaving" whose effect is known for damages to roads, bridges piers and retaining walls. For example, movements in excess of 100 mm (4 in) [9], under basement floors in only three weeks have been recorded. A seven story reinforced concrete frame building on a raft foundation was observed to heave more than 50 mm (2 in).

When the soil is restrained from displacement, the so called "Heaving Pressure" is developed upon soil freezing. Heaving pressure of 1820 kPa. (265 psi) has been recorded [9]. Furthermore, engineers observed that a dramatic loss of bearing capacity of the ground occurred every spring, so that some provinces had to set load restrictions on some of their roads at that time of the year.

1.4 Objective

In the design of stable foundations for cold regions, frost action is a prime consideration, because of the uneven uplift which causes distortion in the structure. Therefore, in the design of soil-steel structures, beside the dead and live loads, the additional loads caused by freezing and thawing of the surrounding soils must be strictly considered. It is very likely that the failures of soil-steel structures often occur due to the lack of proper consideration of frost effect in the design. After studying the present codes for the soil-steel structures, it seems to the author that the effect of soil freezing was completely overlooked especially for the backfill properties. This is in addition to the fact that during construction the codes are some times not respected, which may cause the existence of detrimental particles (silt or clay) in the backfill soil. A complete literature survey for the study of soil freezing and its effect on the roads and highways is presented in the next chapter.

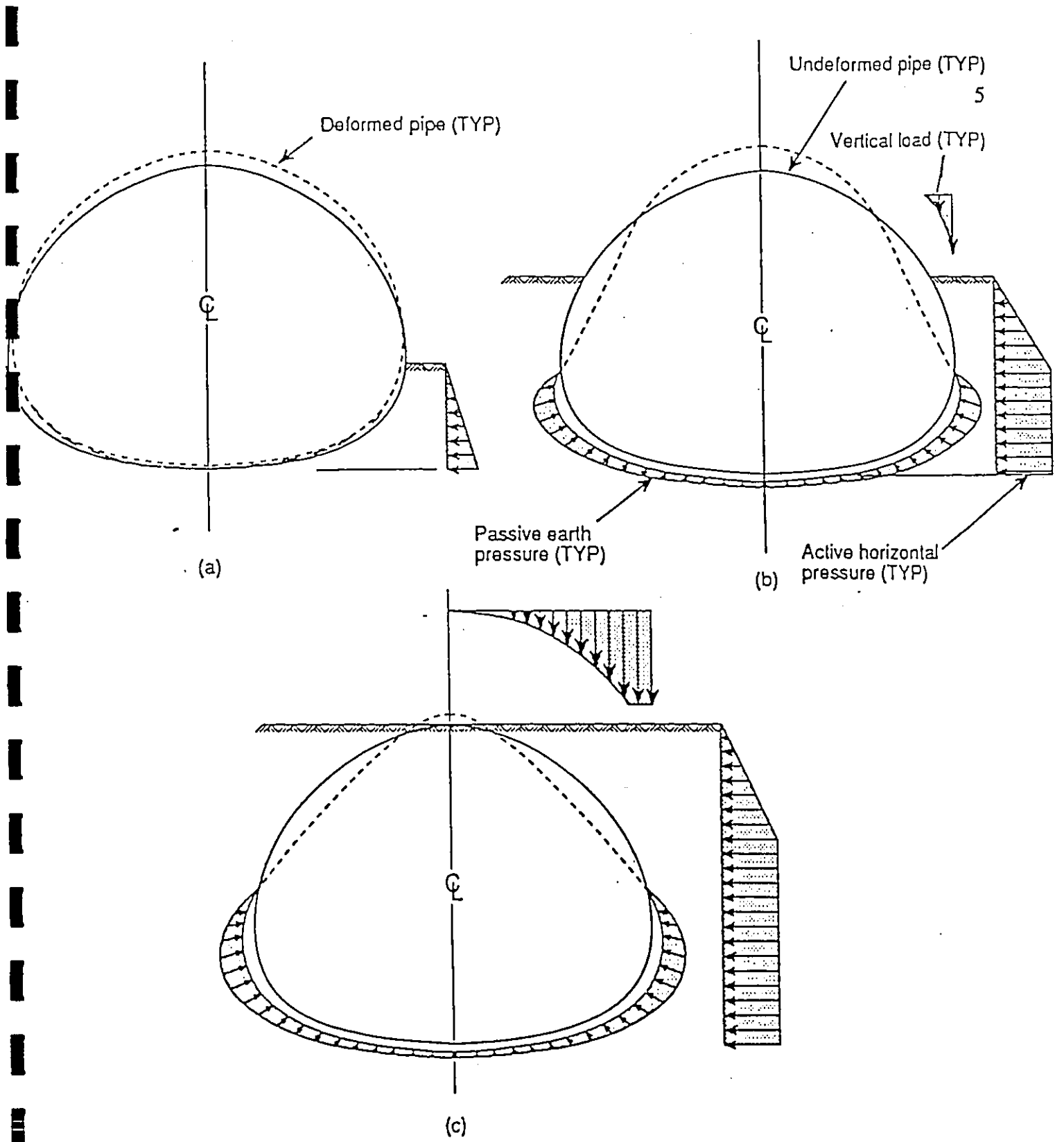
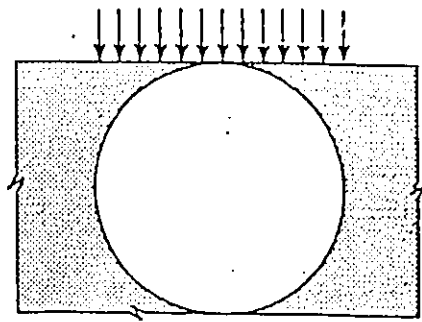
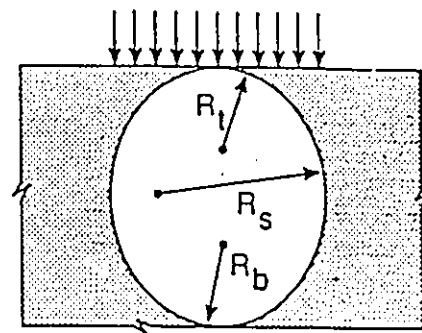
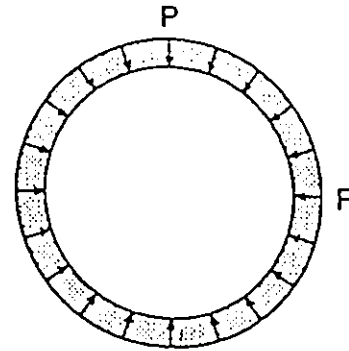


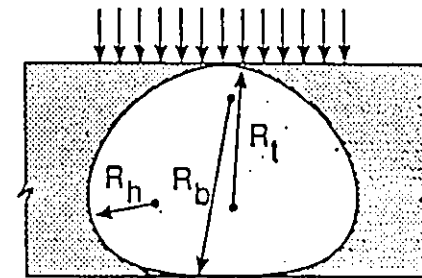
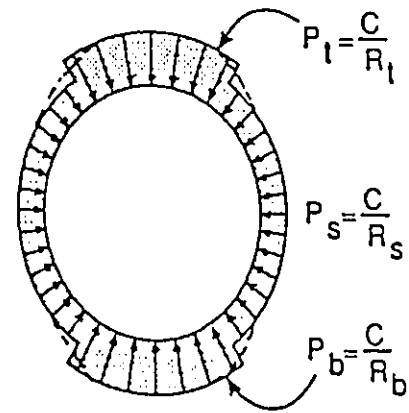
Figure 1.1: Soil-steel structures during various stages of construction (a) fill height below spring line; (b) fill height above spring line; and (c) fill height near crown.



(a)



(b)



(c)

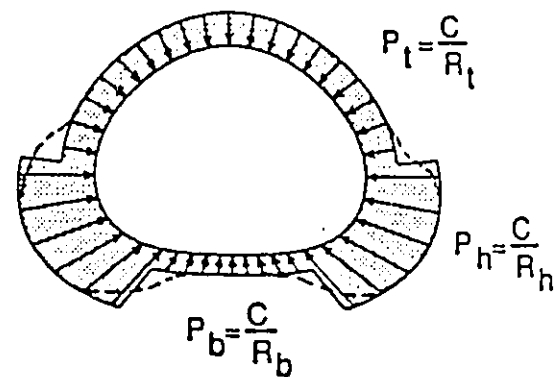


Figure 1.2: Theoretical interface radial pressures around various shapes of soil steel structures (a) round pipe; (b) vertically-elliptical pipe; (c) pipe-arch.



Figure 1.3: Crimps at the haunch of a pipe-arch structure.

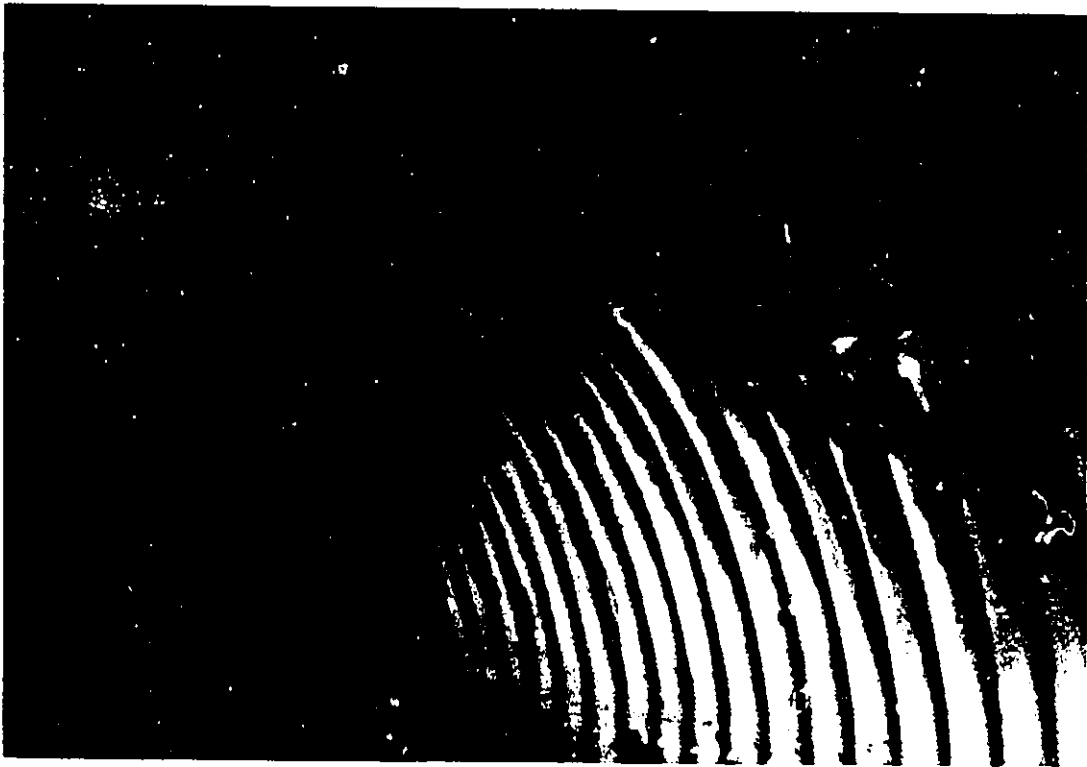


Figure 1.4: Joint rusting in a pipe-arch structure.



Figure 1.5: Bolt tearing at the top of the corner plate of a pipe-arch structure.

Chapter II

LITERATURE SURVEY AND FREEZING THEORY

2.1 Climate

In Southern Canada; the ground freezes every winter and thaws every spring. Engineers observed that freezing of ground may cause very high stresses during winter and loss of bearing capacity during spring. Therefore they tend to place the foundations of the different structures below frost penetration depth to avoid the freezing of the underlying soil. There is a relationship between the air temperature and the depth of frost penetration through the ground. Several equations are available for computing the depth of frost penetration in saturated soil. The most accepted equation is attributed to Stefan [32] :

$$\zeta = \sqrt{\frac{2K_1}{nQ_L\rho_i}(T_f - T_s)t} \quad (2.1)$$

Where

ζ = Minimum frost penetration depth

K_1 = The coefficient of thermal conductivity of frozen soil

T_f = Freezing temperature of water (F)

T_s = Surface temperature (F)

Q_L = Latent heat of fusion for ice per unit weight

t = Duration of freezing (Hours)

$\rho_i = \text{Unit weight of water}$

$n = \text{porosity of soil}$

The U.S. Corps of Engineers experimentally investigated the depth of frost penetration through granular base course under snow cleared airport pavement, in the northern United States. The results of these studies showed a general relationship between the maximum depth of frost and the freezing index (F.I.) which is the cumulative total of degree-days of air temperature below freezing during the entire winter. A degree-day of soil freezing occurs when the average air temperature for one day is one Celsius degree below 0. The relationship between the frost penetration depth and the F.I. is commonly called "Design Curve" as shown in Fig. 2.1 which is used in predicting the frost penetration depth in North America. In case of snow covered areas, the values given in the design curve have to be reduced because of the low thermal conductivity of the snow. Experience has shown, for example, that 0.3 m. of natural snow cover will reduce frost penetration by at least 0.3 m.

Fig. 2.2 is a freezing index map for Canada [4]. Figures 2.1 and 2.2 are valuable information for highway engineers to control that frost susceptible soil is not laying within the frost penetration depth.

2.2 Frost Heaving Mechanism

The process of frost action on soil is an extremely complex problem. It combines geotechnical, heat transfer, and chemistry problems. Sweden was the first country to consider the problem in 1925 after observing serious damages to highways, bridges, piers and culverts. Dr. G. Beskow [7], Swedish Road Institute, Stockholm, devoted most of his life to the study of frost action on soils. The American frost investigations began with

some experiments in 1929 carried out chiefly by Dr. Stephen M. Taber [82], University of South Carolina, and Dr. A. Casagrande [10], U.S. Bureau of Public Roads. All the present researchers refer to these studies as the fundamental background of the frost action on soils.

2.2.1 Structure of Frozen Ground

Beskow (1935), found that there are two different structures of frozen ground as follows:

2.2.1.1 Stratified Frozen Soil

This is the soil in which a part of the pore water is frozen forming large aggregations outside the pores , i.e. layers of stratified clean ice parallel to each other. In clay, for example, the ice layers are thick and widely spaced, built in a uniform and distinct system. In silty soils, they are very thin (a few tenths of a mm.), parallel, and well oriented layers (a few millimeters apart). Generally, it was observed that the coarser the soil, the less is the thickness of ice layer.

Penner (1957) [59] proved that ice lenses are formed normal to the direction of heat flow, and they grow in thickness in the direction of heat flow. This explains the vertical displacement in roads and highways in the direction of the heat flow. On the other hand, the displacement is horizontal when the heat flows horizontally as in case of retaining walls.

2.2.1.2 Massive or Homogeneous Frozen Soil

This is the soil in which all the pore water is frozen and there is no visible accumulation of ice lenses in the freezing zone. This occurs in nature in the freezing of coarse granular soils such as medium sands and coarser soils. Fine grained soils could freeze without forming ice lenses when the water content is well below saturation or when a very quick freezing takes place as outlined later in this chapter. Since this kind of frozen soil has no ice lenses, it usually causes no problems to highways or other structures.

2.2.2 Process of Soil Freezing

The excessive heaving was early attributed to the change in volume of water content in the soil by 9% upon freezing. Taber (1930) [82] showed that the expansion of water due to freezing is not the fundamental cause of heaving. He conducted an experiment in which water was replaced by liquids (Benzene and Nitrobenzene) which decrease in volume during freezing. The test showed strong heaving during freezing provided the freezing soil had access to the liquid during freezing. Furthermore, Beskow [7] showed that heaving of ground caused by the expansion of the existing water, independent of any water supply is under all circumstances very small. Therefore, all the present studies of frost action on soil are mainly concerned with the moisture migration to the soil due to freezing. Hence the process of soil freezing could be outlined in two stages:

2.2.2.1 Ice Crystallization

When a saturated soil is subjected to freezing temperature the water outside the soil pores is the first type of soil water to freeze. It was observed [7] that the ice crystallization began at cracks, fissures, insect and rodent holes since it was the freest

(not-stressed) type of water. However, the remaining does not freeze at 0°C . This water could be described as thousands of thin layers of water molecules surrounding the soil particles. Since these layers are attracted to the soil particle, they require lower temperature, somewhat below 0°C , i.e., extra force to pull the water molecules from the skin of the soil particle and place it in the structure of the ice crystal. The smaller the soil particle and the nearer the moisture film to the soil particle, the more it is attracted to the surface of the soil particle as shown in Fig. 2.3. Thus, a lower temperature is required to freeze all the water films of fine grained soils. While coarse-grained soils freeze homogeneously because all the water in the pores and around the particles freeze at the same temperature (very close to 0°C), fine-grained soil such as clay or silt, freezes discontinuously forming parallel ice layers, leaving the pore water unfrozen between them.

2.2.2.2 Moisture Migration

The decrease in the thickness of water film at a point around the particle, causes a flow of water to that point. This phenomenon is similar to the water flow towards drier soil. Taber (1930) [82] stated "As molecules are removed from the film and attached to the crystal, they are replaced by others from the surrounding water". Therefore, growth of ice lenses results from the migration of water to the freezing zone, from :

1. A high ground water table.
2. Or the still unfrozen portion of the soil.

Since the phenomenon of moisture flow to the ice front is the main reason of the frost action, it is important to elucidate this process accurately. Let us consider a saturated fine grained soil, where the particles are surrounded by uniform films of water, adsorption films. These films do not only act to lower the freezing temperature of the

water, but also they have a mechanical effect, which can be imagined as a pneumatic elastic rubber skin. Water exists at the boundary between the ice and the soil particle, i.e., they do not touch each other directly. If this system is cooled, the water molecules in the film nearest to the ice crystallization freezes and the ice grows downward as shown in Fig. 2.4. This makes the water films around the soil grain thinner, causing an increase in the negative pressure, i.e., pressure deficiency, causing water to flow to this surface. Jumikis (1966) [31] stated that "The main driving force for the water flow to the ice front is the pressure deficiency induced by the lower temperature at the ice front, i.e., by the temperature gradient applied to the freezing soil system. The lower the temperature at the ice front, the greater the magnitude of the driving force". Also, Penner (1957) [59] stated "The freezing point depression at the ice-water interface determines the moisture tension which acts as the driving force during frost heaving".

Hence, the amount of moisture transferred to the ice front is in proportion to the surface area of the soil particles in unit volume. Thus, the smaller the particle size, the greater the amount of water flow to the ice front. It is important to mention here that the process of the moisture transport is a slow one.

2.3 Factors Affecting The Frost Heave

There are many factors that affect the intensity of the frost heave and the formation of ice lenses in soil which are outlined below

2.3.1 Grain Size

The grain size has a double effect because for a very small grain size such as clay, the pores are too small, thus, the resistance to moisture flow becomes considerable and the thickness of adsorption films becomes larger. Therefore, the water flow to the freezing front is decreased. On the other hand when the grain size is large as in gravel, the pores are large, thus, the capillary rise is very small and the thickness of the adsorption films is almost zero. This reduces the amount of water which can be sucked up. There is a complete agreement among all the investigators that silt is the most favourable soil size for the frost heaving since the particles are small enough to provide a comparatively high capillary rise, and at the same time the pores are large enough to allow a quick flow of moisture. Therefore silty soils are the worst frost heaving soils. Several investigators developed criterion of frost susceptibility based on the grain size as outlined in section 2.4.

2.3.2 Rate of Freezing

The process of moisture migration via the films is a slow one. If the rate of freezing is slow, considerable amount of soil moisture can flow from the ground water table or from the unfrozen soil to the ice lenses causing visible ice layers or lenses. On the other hand if there is a sudden drop in the temperature, i.e., quick freezing, moisture flow cannot keep up with the same rate, thus, no ice layers are visible. Beskow [7] found that the formation of ice layers is favored by a very slow lowering of the zero isotherm (isotherm is the line connecting all the points at the same temperature). However, he conducted another freezing experiment on a very fine silt and plotted curve for the time, temperature, and heave as shown in Fig. 2.5 and found that the rate of frost heaving is independent of the rate of freezing. The explanation of this conflict is that if the water

can be sucked up faster than it can freeze, then heaving is proportional to the rate of freezing. On the other hand, if the rate of freezing is greater than the rate at which water can be sucked up, then the rate of heave is independent of the rate of freezing.

2.3.3 Depth of Frost Penetration

There is no relationship between heave and depth of frost penetration. Although greater heave is often associated with greater depths of frost penetration, it is not necessary to have great depth of frost to have great heave, if other conditions are favourable for heaving to occur.

2.3.4 Applied Pressure

Taber [80] noticed that for coarse grained soil a relatively small surface load can entirely prevent frost heaving in an open system while for fine grained soils it did not greatly affect the heaving. Beskow [7] conducted frost heaving test on different soil samples under increasing pressure and the results are outlined in Fig. 2.6. He observed that the increased pressure counteracted the formation of ice strata by squeezing out the adsorption films, thus diminishing the supply of water to the freezing zone. The formation of ice lenses in a freezing soil can be arrested if the frost heaving pressure is counteracted by an equal or greater external pressure.

2.3.5 Compaction

Since the pore size appears to be the major factor governing frost action in soils, anything that alters this parameter will cause changes in frost behaviour. Thus, the degree of compacting which changes the dry unit weight of the soil has an effect on the amount of heaving. Winn and Rutledge (1938) [20] observed that heaving of sandy clay soil increased with the increase in density up to a certain value, beyond which heaving

decreased with further increase in density. Clearly, for each soil there is one arrangement of the soil particles that causes the most favourable conditions for frost action to occur.

2.3.6 Degree of Saturation

When the soil is not fully saturated, the pores are filled by water and air which means that the air pores are providing enough room for free expansion of the water. Beskow [7] explained this by Fig. 2.7 where, (a) is the soil particles before freezing, (b) the freezing caused the soil to expand much greater than 10% increase due to water freezing, or the ice crystals may grow in the free space of the pores as shown in (c) hence, noticeable expansion could occur with non-saturated soil, however a small pressure 5 to 10 kpa (0.7 to 1.4 psi) will reduce the expansion to zero because the ice will grow in the free space. In general, frost heave increases with the increase in the degree of saturation. That is why frost action is normally at its worst case in a winter following a very wet fall.

2.3.7 Thawing and Refreezing

There is a complete agreement among investigators that frost action is usually more severe after a thaw than after a single freeze, and that there is a greater increase in soil moisture content and reduction in load bearing capacity following several cycles of freezing and thawing. Taber [82] explained this phenomenon and stated "The effects of refreezing after a thaw are also accentuated by the fact that the first freeze leaves the soil in a more or less loosened or expanded condition."

2.4 Criteria of Frost-Susceptibility

It is important to recognize that not only the proper conditions of temperature and water supply are necessary to cause frost action in soils but also the type of soil involved. From the design and construction standpoint, it is important to be able to recognize which soils are frost-susceptible, i.e., subject to the formation of ice lenses in sufficient number and of appreciable thickness to produce detrimental heave effects during winter and loss of bearing capacity during spring. The key to the frost action problem in engineering is to be able to establish the frost susceptibility of any soil. Thus, there was a need for a criterion to determine to what degree a given soil will be frost-susceptible soil without conducting experimental test. Several investigators have sought to develop relationship between soil characteristics and the intensity of frost action. The most commonly used criteria in North America are outlined below.

2.4.1 Beskow's Criterion

Beskow [7] investigated 9 saturated samples of pure fraction (poorly graded) of different grain sizes. He found that no stratification occurred above an average diameter of 0.1 mm. He established a relationship between capillarity and frost-susceptibility for both moraine soils (well graded) and sediments (uniformly graded) as shown in Table 2.1. Beskow noticed that for sandy soils the frost heaving is of no importance at all because, for unloaded soil it can reach 10 mm maximum heave, and for a load of about 10 kpa (1.4 psi) or more it is reduced to zero.

2.4.2 Casagrande's Criterion

By far the most commonly used criterion for frost-susceptibility is that of Casagrande (1931) which is based on field and laboratory studies. He stated "Under natural freezing condition and with sufficient water supply one should expect considerable ice segregation in non-uniform soil containing more than 3% of grain smaller than 0.02 mm and in very uniform soils containing more than 10% smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1% of grains smaller than 0.02 mm even if the ground water table was as high as the frost line". Twelve agencies have contributed to a survey of Frost Design Practice in Canada (1963) [4], including the Federal Department of Transport and Public Works, and the Highway Departments of the ten Provinces. They noticed that when the silt content of granular materials reached 6 to 8 percent, and the moisture supply was ample, heaving was significant and thawing was accompanied by loss of density and bearing capacity during the spring. Furthermore, the U.S. Corps of Engineers has done extensive research on frost action on roads and airfield pavements in seasonal frost areas since 1945. They reported [6] a good correlation between heave rate and the percentage of particles finer than 0.02 mm, which confirm Casagrande's criterion.

2.5 Heaving Pressure

Heaving pressure is the pressure developed upon the freezing of soil when the soil is restrained from displacement. Migration of the moisture to the freezing front causes heave and/or heaving pressure. According to a report from the National Research Council, Canada, [9] there is no apparent limit to the amount of heaving pressure that may occur upon the frost action in soil. Heaving pressure of 1.82 MPa. (265 psi) has been measured.

There is a complete agreement among the researchers that if the external pressure is equal to or greater than the heaving pressure a state of equilibrium will exist and the formation of ice lenses can be arrested. Therefore, it is important in the design to be able to predict the magnitude of the heaving pressure of any soil. A brief discussion of the reasons for the heaving pressure is necessary for understanding the theory of heaving pressure prediction.

2.5.1 Reasons for The Heaving Pressure

As mentioned in 2.2.2.2 the temperature depression is a very important condition of the ice lensing process because it provides the energy for the development of heaving pressure. The following equation is applicable [61] in a freezing soil system to specify freezing temperature change caused by of curvature of the ice front (Penner, 1959)

$$dT = \frac{2\sigma_{iw} T_m}{\rho_i Q r} \quad (2.2)$$

Where

dT = Freezing point depression

σ_{iw} = ice-water interfacial energy (ergs/cm²)

r = radius of the ice water interface

ρ_i = density of the ice (gm/cm³)

Q = latent heat of fusion (ergs/gm)

T_m = temperature of melting at zero curvature of the solid liquid interface (°K)

If the quantity r in the above equation interpreted as the radius of the pore, the smaller the pore, the lower is the temperature necessary before the ice front can advance. This means supercooling of the pore water below a growing ice lens. Thus, the freezing

temperature of the ice water interface is higher than that of the pore water. The subsequent release of energy causes: (1) creation of a moisture tension gradient which generates moisture flow to the ice front, (2) developing of a positive pressure to raise the overburden soil and providing a space for the ice lens to grow. The relationship between the pressure and the freezing point depression in case of saturated system is given by Edlefsen and Anderson [59] as follow:

$$dP = \frac{dT \cdot Q}{(V_w - V_i)T} \quad (2.3)$$

Where

dP = total change in pressure

Q = Latent heat of fusion

V_w = specific volume of water

V_i = specific volume of ice

T = the absolute temperature at which the phase change occurs

dT = change in freezing point temperature

Penner [60] explained that applying the pressure dP to the system causes the freezing temperature of the pore-water and that of the ice water interface above the soil particle to be the same. Therefore no moisture flow can be induced or positive pressure can be developed. Thus, applying the pressure dP will stop ice lensing. This fact is of a great practical importance. Beskow [7] proved this fact by applying different pressures on saturated soil system during freezing and recording the corresponding heave as shown in Fig. 2.6. He also noticed that the capillary pressure has the same effect as the load effect on the frost heaving. Penner and William [63,87] agreed that the growth of ice lenses can be stopped by applying sufficient load on the soil or by applying suction to the water in

the pore system, i.e., by lowering the ground water table or combination of both. Hence, when the water table is high (low capillary pressure), conditions for frost heaving are favorable. This explains the significance of the level of the ground water table.

From the above equations it could be outlined that the prediction of the susceptibility of a given soil has to be based on the pore size more than the grain size.

2.5.2 Heaving Pressure Prediction

Attempts to find theoretical prediction for the heaving pressure of granular soils have been made by many investigators. It has been agreed that an analogy exists between the phenomenon of the water rise in capillary tube and the moisture suction at the frost line. This concept is briefly described as following:

When an open ended glass capillary tube is placed vertically with its lower end in the water, water will rise or sucked up to a certain height, which is the capillary rise. The interface, the meniscus, between the water and the air is curved. There is an associated difference in pressure between the air and the water at the interface, the water having a lower pressure relative to that of the air which causes the water to rise up, until the weight of the water column is equal to the lowered pressure. The difference in pressure is greater, the greater the curvature.

Physicists and chemists agreed that the pressure difference phenomenon is not limited to the interface between air and water but the same phenomenon occurs at the interface between ice and water. They found that the pressure difference between water and ice can be expressed as:

$$P_i - P_w = \frac{2\sigma_{iw}}{r_{iw}} \quad (2.4)$$

where

P_i = pressure on ice, overburden pressure

P_w = pressure in porewater

σ_{iw} = ice-water interfacial energy or surface tension

r_{iw} = radius of the ice-water interface

Although the above equation seemed easy to predict the heaving pressure, there were two problems facing all investigators using this equation in the determination of: (1) the internal pore geometry of the porous material, r_{iw} , and (2) the exact value of the ice water interfacial energy. The papers by Everett and Haynes [16], Penner [62], and Williams [87] represent the most significant studies of these problems therefore they are outlined herein.

2.5.2.1 Everett and Haynes (1965)

Everett and Haynes assumed a close-packed array of spheres of equal radii Fig. 2.8. They tried to find a relation between the radius of the particle and the radius of curvature of the meniscus passing through the aperture in the closed packed spheres. This value was examined by other researchers at that time, their results ranged between 3.19 and 9.75, while Everett and Haynes, suggested a value of 5.6 based on their experiment of the capillary rise between three close-packed cylindrical rods. Their final equation for calculating the heaving pressure was as follow:

$$dP = \frac{2\sigma_{iw}(1+B)}{r} \quad (2.5)$$

Where

dP = heaving pressure

σ_{iw} = interfacial energy between ice and water

r_{iw} = radius of curvature of meniscus passing through

aperture in close-packed spheres

r = the radius of the spheres

$$B = \frac{r}{r_{iw}}$$

2.5.2.2 Penner (1966)

Penner measured the pressure resulting from ice lens growth on pads of glass beads, unconsolidated material, to test the equation of Everett and Haynes. The glass beads were size fractionated by sedimentation in water to obtain size uniformity. Two water saturated specimens were used (1) size varied from 9 to 19 microns in diameter having 80% between 10 and 15 microns and (2) size varied from 15 to 27 micron, having 80% between 17 and 22 microns. Penner used the median diameters for both samples, 12.0 and 19.4 microns, respectively, and a value of 35 ergs/sq.cm for the ice water interfacial energy in the calculation according to equation (2.5). The measured points are given by closed squares in Fig. 2.9. The remainder of the points were obtained using non-uniform fractions of natural soils and Potter's flint, but in each case the radius used for plotting was the smallest for that particular sample. Consequently, Penner considered that the Everett and Haynes model was correct and valid.

2.5.2.3 Williams (1966)

Williams referred to the pressure difference between water and ice, equation (2.4). He defined P_i as the pressure on the ice, equal to the total pressure, and P_w as the pore water pressure at the interface when the frost line is penetrating through the pores. When the temperature falls, the ice lens advances through smaller and smaller opening viz.

smaller ice water interface. In accordance with equation (2.4), an increase in the pressure difference between the water and the ice is developed. In another words, the value $P_i - P_w$ must then be equal to that given by $2\sigma_{iw}/r_{iw}$. This occurs only if the pore water pressure decreases, which brings about the migration of water to the frost line. He stated " The migration of water occurs as a result of the hydraulic gradient set up a fall in pore water pressure at the lower boundary of the frozen layer.". Williams considered equation (2.4) was the boundary between stratified and homogeneous frozen soils. However, depending on pore water pressure (water table location), confining pressure and the value of r_{iw} , one of three conditions could occur:

$$P_i - P_w < \frac{2\sigma_{iw}}{r_{iw}}$$

which means the migration of water to the ice front i.e. frost heaving soil,

$$P_i - P_w = \frac{2\sigma_{iw}}{r_{iw}}$$

which means that the ice lens ceases to grow i.e. non-frost heaving soil,

$$P_i - P_w > \frac{2\sigma_{iw}}{r_{iw}}$$

which means that water will be expelled away from the soil, this will be discussed later in this chapter.

This principle explains the structure of the permafrost areas where they are usually divided into an upper, ice-rich layer containing ice lenses, and a lower part free of lenses, while the pressure difference $P_i - P_w$ is greater at the upper part than that at the lower part.

2.5.3 Application of Heaving Pressure Theory

When applying equation (2.5) to predict the heaving pressure of a given soil, the following points should be realized with regard to the presented theory:

1. There is a general agreement that the heaving pressure reaches a maximum value when the frost line become stable, i.e., when no further frost penetration occurred. According to a survey of frost design and construction practices supplied by Canadian highway engineers, most of the heaving seems to occur in the latter half of the winter season, probably when the frost line has reached its maximum penetration.
2. Equation (2.5.) is thought to predict the heaving pressure due to ice lens growth at the freezing plane only, i.e., at the interface between the frozen and unfrozen soil. Penner, in a discussion with Yong and Osler (1971), [66], noted that the freezing of water trapped in the frozen zone may develop stresses, but this eventuality was outside the consideration of the theory and not relevant to the model. He considered these stresses to be relatively small.
3. Although equation (2.5) seems to be reliable, there is specific argument about the value of r (the effective radius of the soil) in case of unsorted soils. However, Penner used the size of the smallest particle in the sample regardless of the percentage of that size or the grain size distribution. Some other researchers suggested the value of D_{10} as the effective diameter regardless of the degree of compaction.
4. Yong and Osler (1970) [92], found that the generated heaving pressure increased with the increase of the degree of constraint. They conducted three experiments using saturated soil sample which was subjected to uniaxial freezing in open

system. The freezing sample was forced to thrust against proving rings of varying stiffness. The largest heaving pressure was generated by heaving against the stiffest constraint.

5. The researchers did not agree on the value of the ice-water interfacial energy. Miller et al. (1960) [54] used a value of 17 ergs/sq.cm, Penner (1966) [62] used a value of 35 ergs/sq.cm and Yong (1966) [91] used a value of 20 ergs/sq.cm

Subsequently, it must be emphasized that a precise prediction of frost heaving pressure in practice is not possible in all cases. Penner (1969) [65] stated "The magnitude of the heaving pressure at the freezing plane cannot be reliably predicted for most natural soils". Also, Chapuis (1988) [11] stated "The uplift forces cannot be predicted accurately but they are important. Most structures are not designed to resist these forces and the usual engineering solution is to avoid this problem by three means : drainages, insulation, or sufficient soil thickness of non-frost susceptible soils"

2.5.4 Application of Heaving Pressure Theory on Casagrande's Criterion

Let us consider a poorly graded soil (uniformly graded) with 10% by weight finer than 0.02 mm, i.e., a soil just on the borderline of considerable ice segregation soil as classified by Casagrande in 2.4.2. Assuming the average diameter is 0.05 mm and using a value of 35 ergs/sq.cm for the ice water interfacial energy as was recommended by Penner [62, 63]. According to equation (2.5) a heaving pressure of 18 kPa (2.6 psi) is obtained.

For a well graded soil with 3% by weight finer than 0.02 mm, according to the gradation characteristics, D_{10} as the effective diameter would be about 0.04 to 0.05 mm. Thus, a heaving pressure of 20 kPa (2.9 psi) is obtained. The frost penetration is usually

1.0 to 1.2 m, in a soil of unit weight range from 16.0 to 18.6 kN/m³ (100 to 120 lb/cu.ft.). Thus the overburden pressure at the freezing plane is about 19.5 kPa (2.85 psi) which is enough to arrest the formation of ice lenses. If the soil contains fine particles more than what Casagrande defined, the overburden pressure will not be enough to prevent the formation of ice lenses. Thereafter, considerable ice segregation could be observed.

2.6 Water Expulsion During Freezing

Referring to equation (2.4) which was described by Wissa and Martin (1968) [43] as nothing more than a straight forward application of the effective stress principle, McRoberts and Morgenstern (1974) [43] suggested that if the pressure difference across the freezing front is greater than $2\sigma_{iw}/r$, ice will propagate through the pores of the soil and water may be expelled away. This case could occur when the applied pressure is high and/or when r_{iw} is large. Therefore, a condition favouring expulsion of water exists when saturated coarse grained soil (gravels or sand with small amount or no fine materials) is frozen in an open system. Wissa and Martin found that when saturated Ottawa Sand (uniformly graded with no fine materials) was frozen in a closed system with allowed free axial expansion, heave and positive pore pressure were recorded. When an identical sample was frozen in an open system, water was expelled from the sand during freezing. Hoekstra (1969) [24] found that for a saturated granular soil in which little or no consolidation takes place upon freezing, the constant volume condition requires that some water has to be expelled from the sample when the ice forms. The amount of water that will be expelled is about 9% of the pore volume. Thus, the water content in the frozen part in granular soil is reduced by about 9%. Fine grained soils may also expel water when a suitable surcharge is applied. Arvidson (1973) [5] conducted

freezing test on Devon Silt in an open system. He found that there was a critical pressure C given by :

$$C = \frac{2\sigma_{iw}}{r} \quad (2.6)$$

When the total stress on the freezing soil is lower than C , water was attracted to the freezing front. When the total stress exceeds C water was expelled away from the freezing front. This pressure C was termed as the "Shut-off" pressure. The definition of this term according to McRoberts and Nixon (1975) [44] is "the effective stress at the frost front which will cause neither flow of water into or away from the freezing front".

The water expelled away from frozen soil in an open system flows through the unfrozen soil beneath the freezing front. Depending on the rate of expulsion of the pore water and on the permeability of the underlying soil, the pore water pressure will increase. If the permeability of the underlying soil is zero, i.e., the expelled water cannot be forced into the soil, the pore water pressure is instantaneously made equal to the overburden pressure. This reduces the effective stresses of the underlying soil which results in reduction in the soil shear strength. Therefore, it is possible that unsafe conditions could be created.

2.7 Frost Boil

When frost-susceptible soil freezes under the proper condition of water supply, the formation of ice lenses occurs causing frost heave. The amount of surface heave is approximately equal to the thickness of the ice lenses. Toward the end of the winter season or in early spring, upon thawing, the water released from the ice lenses will cause local over-saturation until the excess water can leave the thawed zone. In spite of the fact that in an unfrozen state, the soils could not have much more than 30% water content,

Kokkonen (1926) [7] has found that the water content in some soils increased up to 300% by weight upon freezing. This occurred in the form of clean thick ice layers. Since thawing commences from the surface and progresses inward, the still frozen zone acts as an impervious diaphragm, tending to trap the excess water in the upper layer.

The increase of the water content of the soil causes a decrease in the soil density and high excess pore water pressure which brings about the reduction of the shear strength of the soil. Therefore, until the soil has completely thawed, a condition of super-saturation often exists in frost susceptible soils which is characterized by dramatic settlement and loss of supporting capacity of that soil. Seasonal loss of supporting capacity of road and airfield is a well known example of this effect. This phenomenon is called "Frost Boil".

Ice segregation during freezing is not the only cause for ground weakening, especially in fine grained soil. The increase in water content may be caused by the precipitation which is blocked of drainage because of the still frozen soil. Accordingly, frost boil may occur even in non frost susceptible soil.

Actually, the thawing of frozen ground takes place from both upper and lower surface of the frozen stratum, because the ground water table and the air temperature are the heat sources. If the air temperature is well above 0°C the melting occurs from the surface, and the water set free cannot escape downwards through the soil voids until thawing is practically complete within the entire frozen layer. On the other hand, if the air temperature remains barely below freezing for a long time, a deeply frozen soil will thaw gradually from the bottom and the water may then return to the ground water table. The U.S. Corps of Engineers made numerous observations of ground thawing on roads and highways. It was found that thawing usually progressed faster from the upper surface. Also, several authors have written that damages to roads is most severe when

the thaw is sudden and rapid from the upper surface downwards, causing the melt water to be released quickly and not be able to drain. In a survey of Frost Design Practice in Canadian Roads, 1963 [4], it was reported that a reduction in load carrying capacity of the order of 40% in Alberta, 50% in Ontario, and 30 to 60% in New Brunswick occurred. The effect was worse on flexible pavement than on rigid pavement. That is why some provinces had to set load restrictions on some of their roads in the spring period.

Furthermore, the U.S. Corps of Engineers (1952) [28] has shown that the wheel load supporting capacity of a flexible pavement during the frost melting period may be of the order of one third of that of normal period. Rigid pavements, on the other hand, have been found to retain about three quarters of their normal period wheel-load supporting capacities. The smaller reduction in the strength of rigid pavements due to frost action is attributed to the fact that the supporting capacity of rigid pavements are not influenced by changes in the subgrade strength to the same degree as flexible pavements due to shear deformation and remolding during the critical spring frost-melting period. Therefore the Corps of Engineers specifies a non frost-susceptible soil for the base-course material for a depth not less than the depth of frost penetration; otherwise the design must be based on the reduction in the strength of the subgrade during the frost-melting period.

Nevertheless, Jumikis (1956) [30] has done extensive research to find the effects of frost action and the loss of bearing capacity of 30 New Jersey soils. It was found that the soils A-1-a, A-1-b, and A-3 showed the least amount of relative heaving and loss of bearing capacity. However they showed a loss of bearing capacity ranging from 10% to 25%. They were considered the best compared to the other soils. Soil A-2-4 showed a loss of bearing capacity ranging from 25% to 50%.

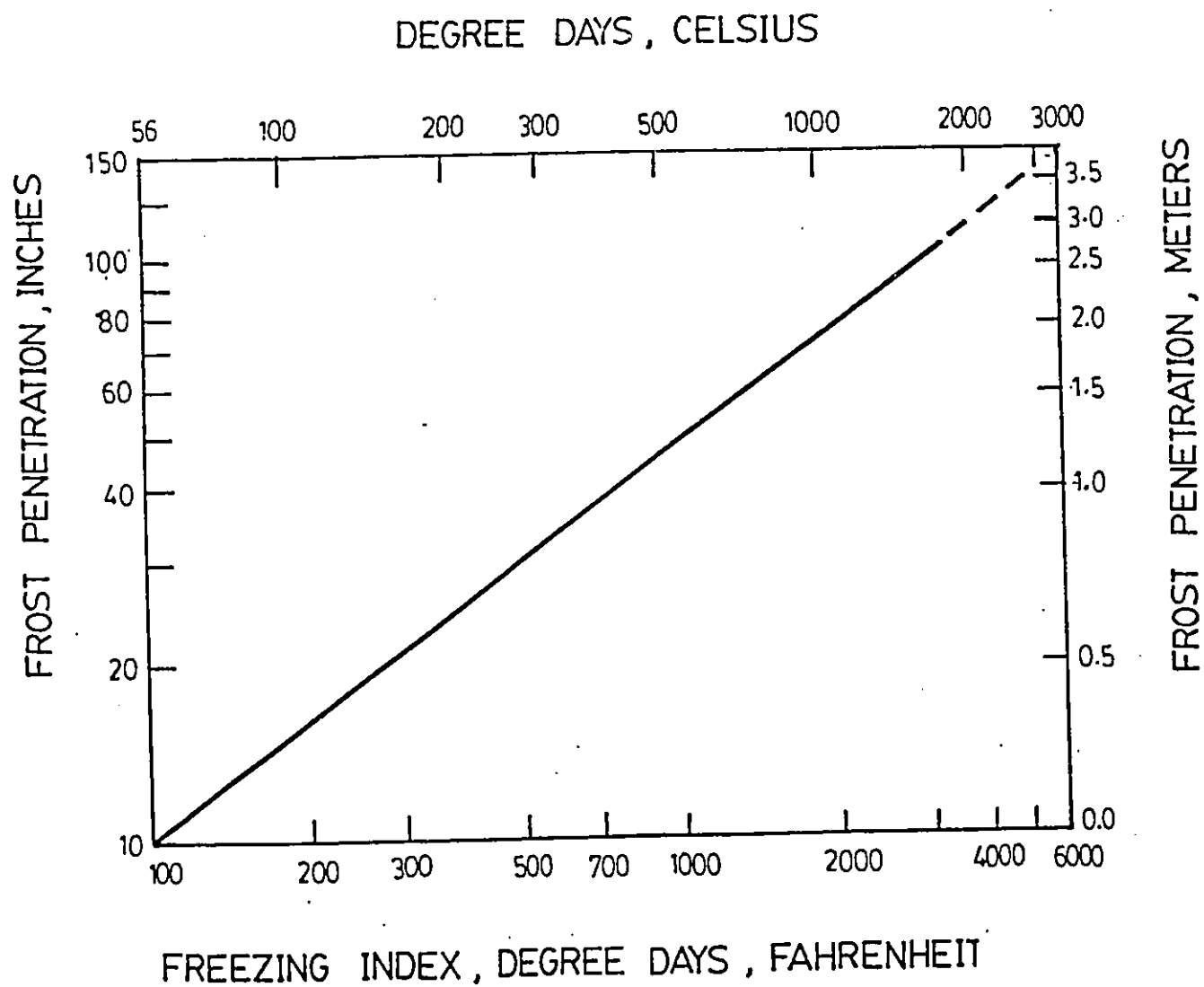


Figure 2.1: Relationship between Freezing Index and depth of frost penetration.

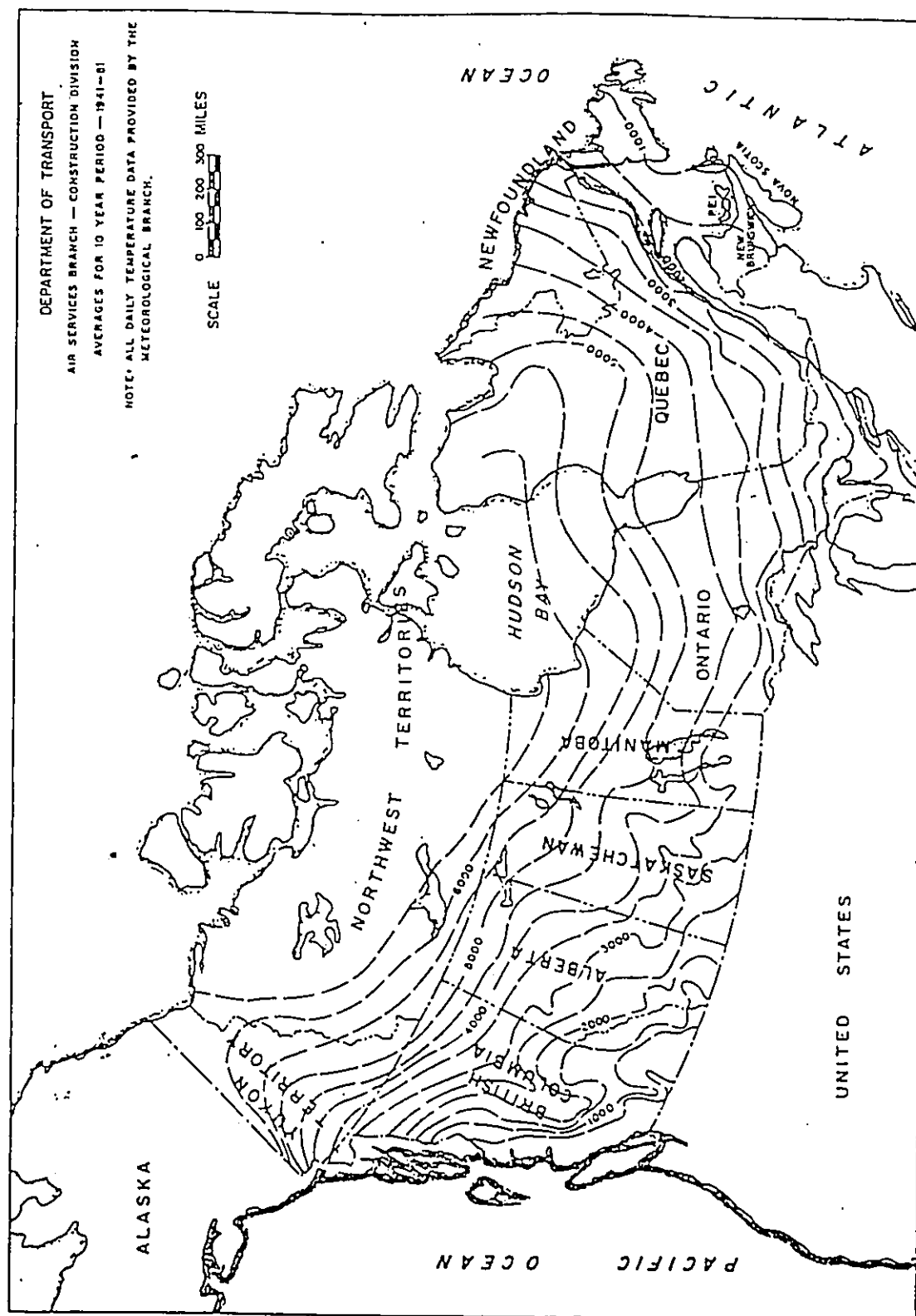


Figure 2.2: Freezing Index map in Canada.

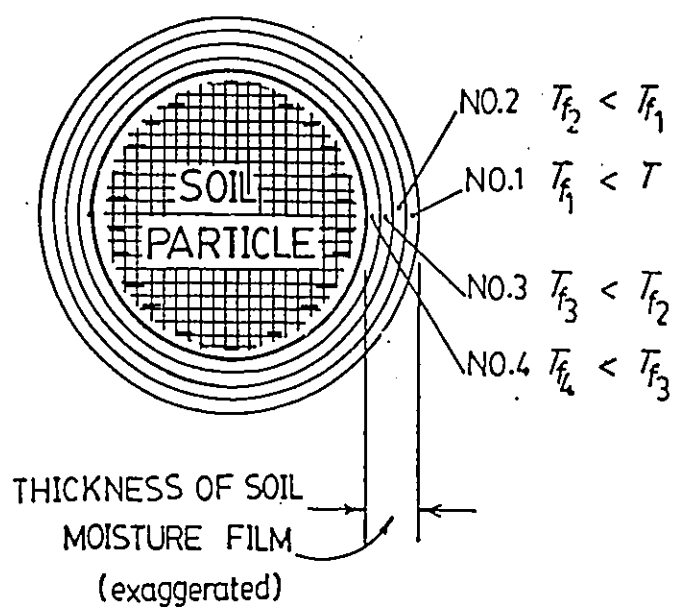


Figure 2.3: Freezing of soil moisture.

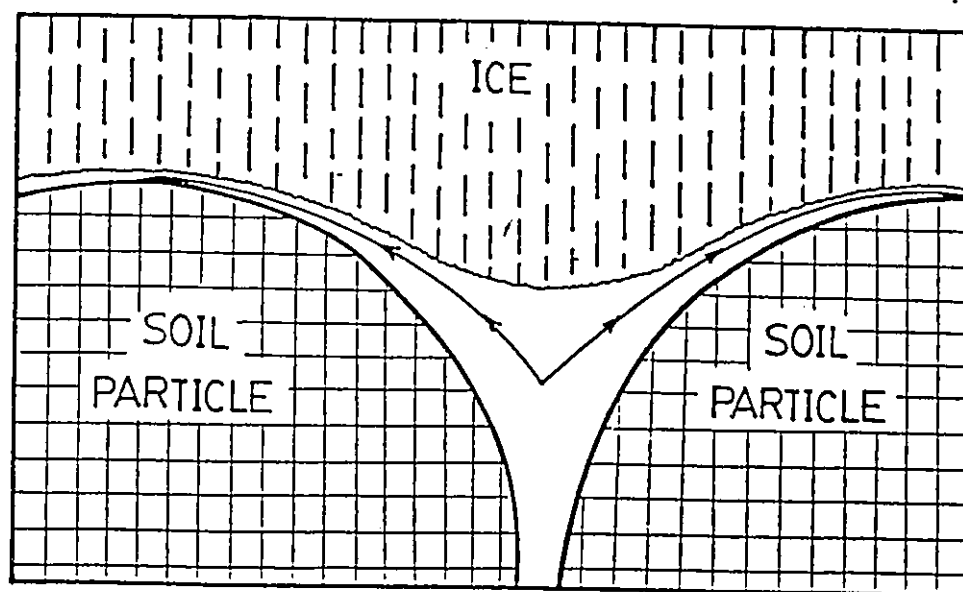


Figure 2.4: The boundary between growing ice-crystal and the underlying soil particles.

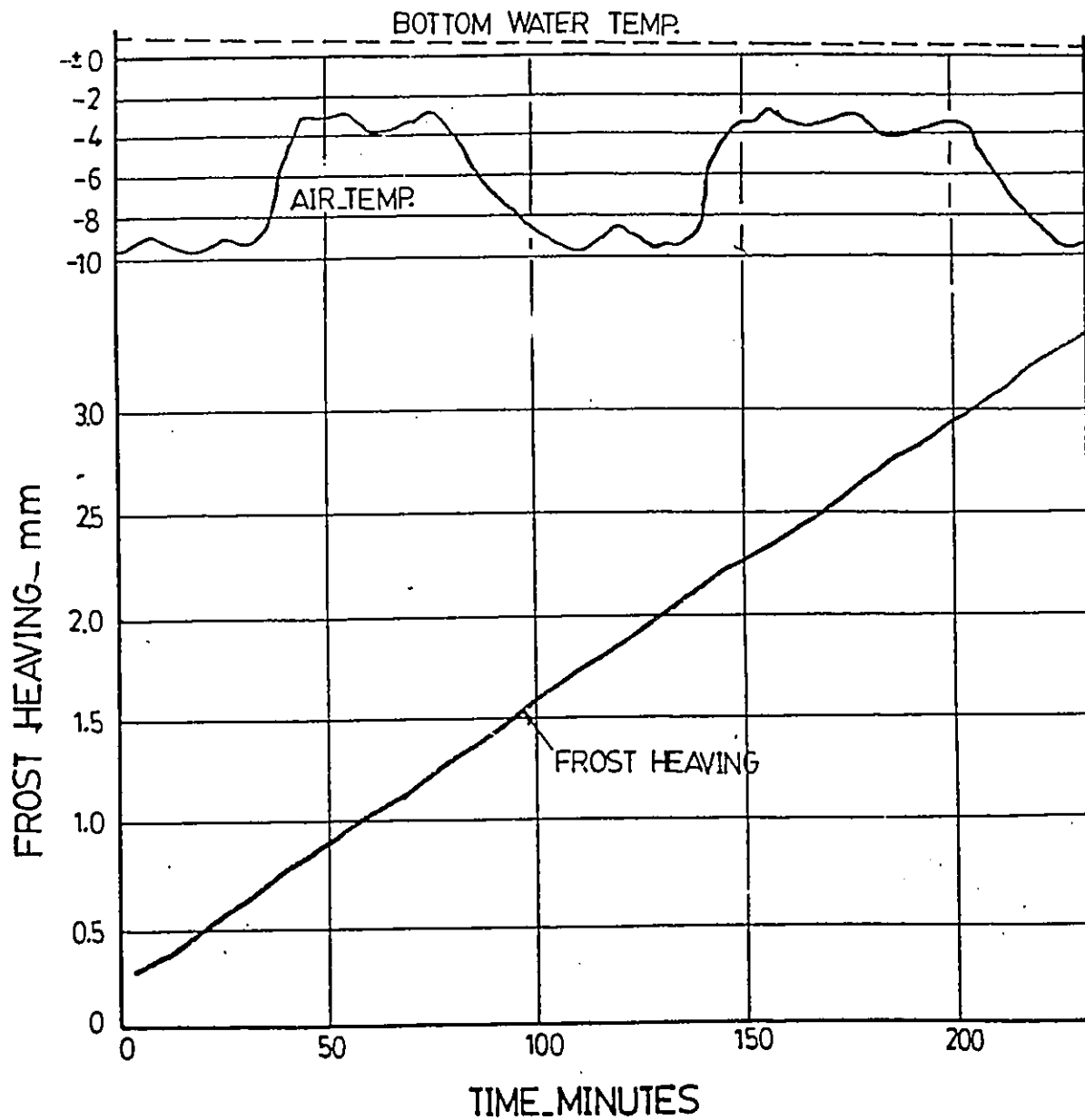


Figure 2.5: The relationship between frost heaving and rate of freezing (Beskow 1935).

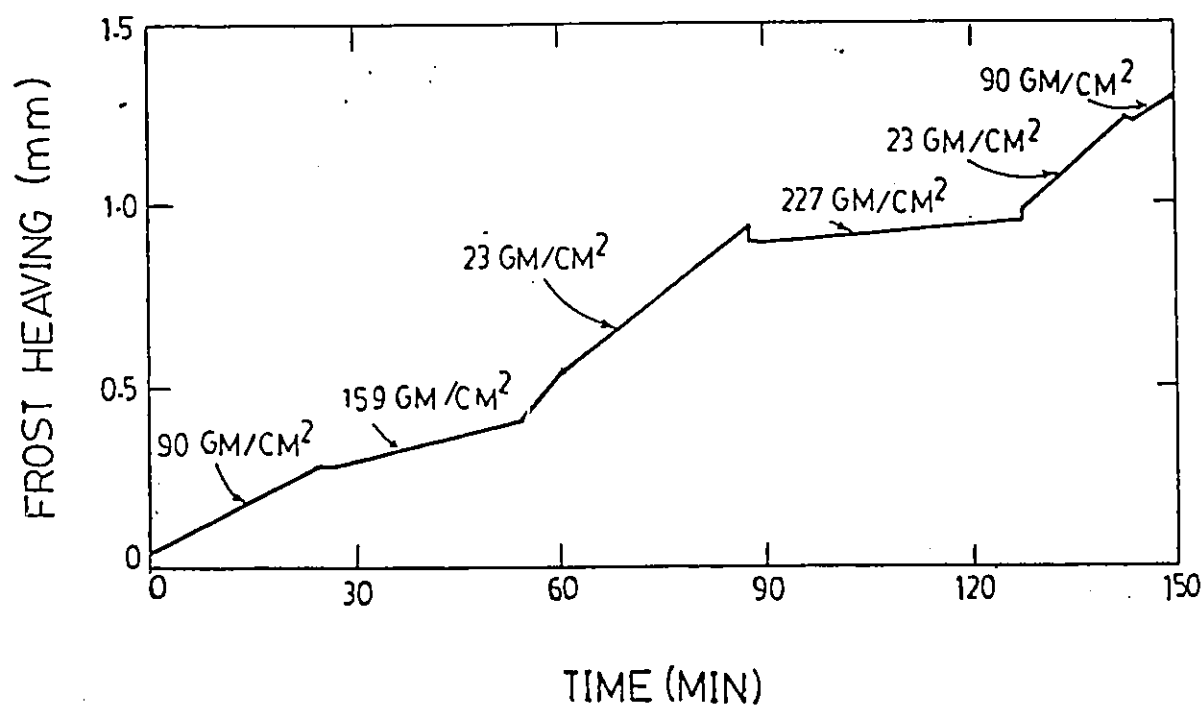


Figure 2.6: The effect of applying pressure on frost heave (Beskow 1935).

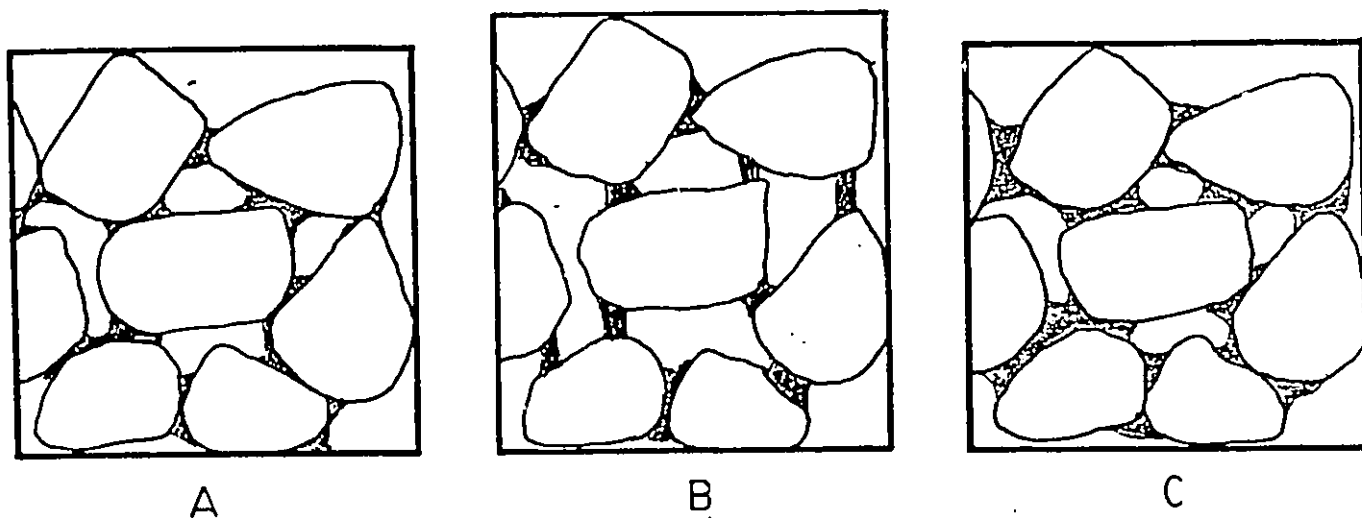
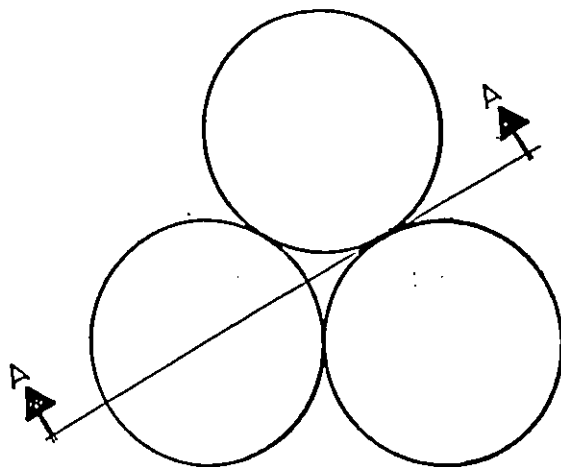
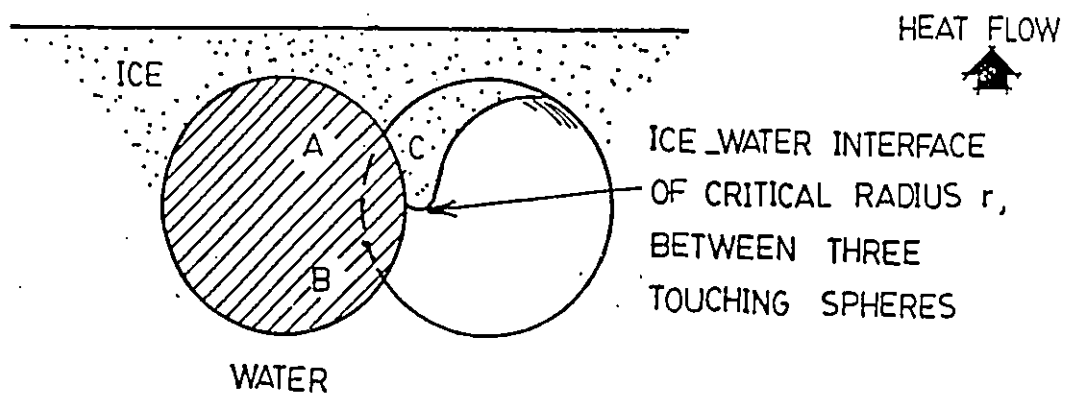


Figure 2.7: Freezing of non-saturated soil (A) soil particle before freezing; (B) expansion more than 10% when there is no pressure; (C) any small pressure will cause the ice to grow in the free space.



(a) SPHERES IN CLOSE-PACK ARRAY



(b) SECTION A-A

Figure 2.8: Everett and Haynes Particles Model.

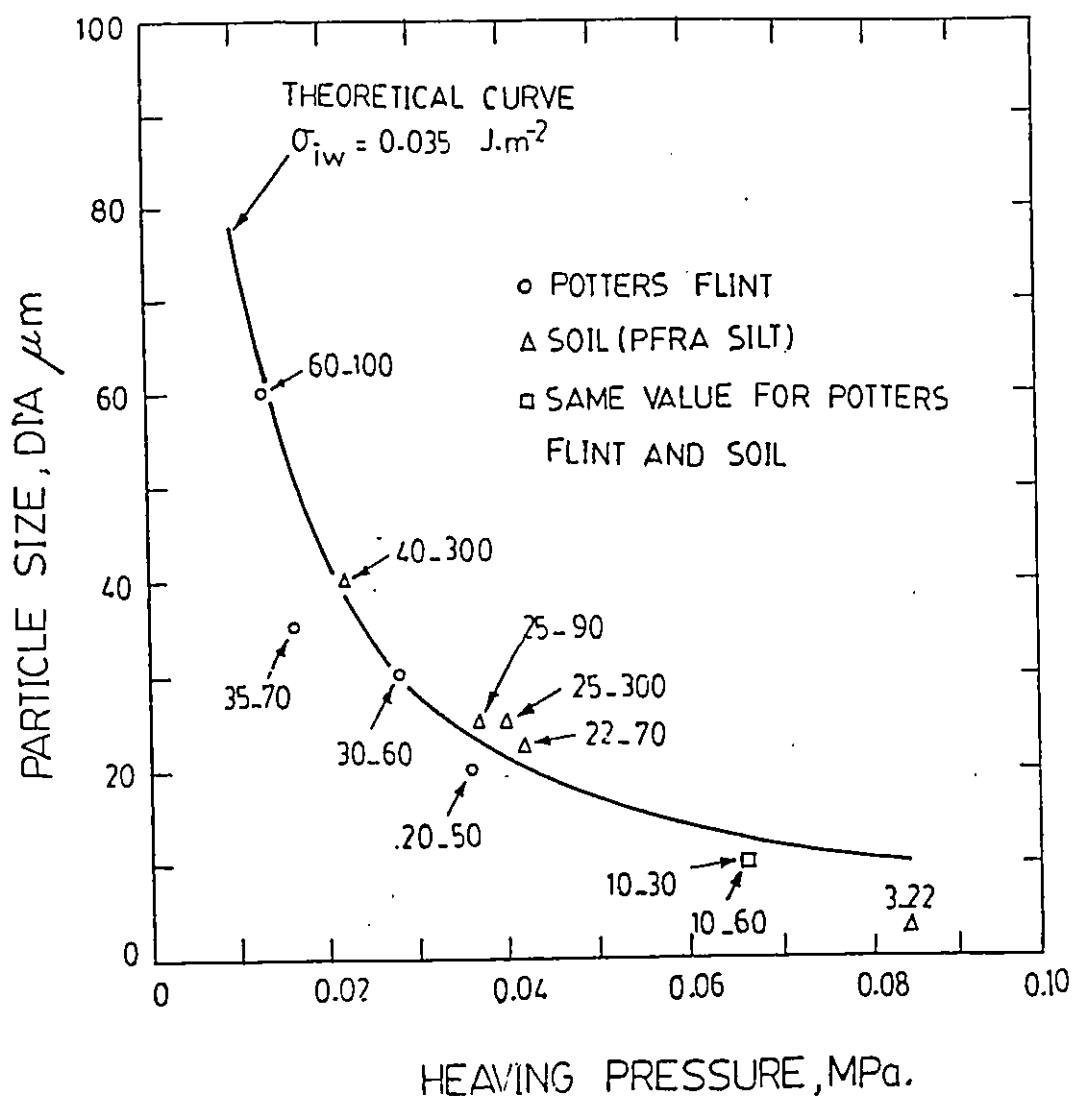


Figure 2.9: Relationship between experiment and theoretical heaving pressure.

Soil Group		Average Diameter	Amount Passing Sieve		Capillarity K
		mm	0.062mm	0.125mm	meters
0. Non-frost-heaving under any circumstances	Sediment Moraine	0.1 -	<30% <15%	<55% <22%	<1 <1
1a. Causing frost-heave only at surface and for very high ground water	Sediment Moraine	0.1-0.07 -	30-50%	-	1 - 1.75
1b. Same, excepts effect whole road base for very high ground water	Sediment Moraine	0.08-0.05 -	15 - 25%	22-36%	1.25-2.50
2. Normally frost-heaving & liable to frost boils for ground water depths 1-1/2 m (moraine 1m)	Sediment	<0.05	>50%	-	2 - 20
	Moraine	-	>25%	36%	
3. Frost-heaving clays but not liable to boils	Sediment	-	-	-	20-?
4. Non-frost-heaving stiff clays	Sediment	-	-	-	?

Table 2.1: Beskow's Criterion of Frost Susceptible Soils.

Chapter III

FREEZING EFFECT TO SOIL-STEEL STRUCTURES

3.1 Application of The Soil Freezing Theory on Soil-Steel Structures

The study of the frost action effect (freezing and thawing) on roads and highways (chapter II) can be applied to analyze the effect of this phenomenon on soil-steel structures and their durability. Herein, it must be recognized that prior to freezing, water may find its way into the fill soil around the culvert and accumulate there in several ways. It may seep into the soil from precipitation, from embankment slopes of fills and cuts, or from high ground water table. When the ground water table is lower than the water in the structure, it may enter the soil through side walls of the culvert since the plates of soil steel structure are not water proof at plate laps and bolt holes. Hence, at the fall, the soil around the culvert could be saturated. This is true especially when the original soil below the conduit has such a low permeability that slows the drainage of this water.

3.1.1 Thermal Analysis

In light of the study of frost action on roads the temperature profile of soil-culvert interaction could be visualized as shown in Fig. 3.1. This is irrespective of their cross section, whether circular, elliptic, or pipe arch. If the backfill soil is frost susceptible and that culvert is subjected to a freezing temperature, the thermal system around the culvert may be described as follows:

3.1.1.1 During Freezing

1. During freezing period the inside space of the culvert contains cold air. Thus, heat flows from the region of higher temperature (the backfill soil), to that of lower temperature (atmosphere), causing temperature gradient parallel to the heat flow. The direction of the heat flow is perpendicular to the cooling surface, which is the culvert wall.
2. Temperature gradient brings about the frost to penetrate the soil fill around the culvert through the walls forming a ring of frozen soil of thickness equal, approximately, to the frost penetration depth at this area (as was discussed in 2.1.).
3. The phase change of the pore water to ice increases the moisture suction at the 0°C isotherm and draws in water from the unfrozen soil or from the ground water table causing, eventually, "Frost Action" as discussed in 2.2.1
4. Upon frost action in the soil fill around the culvert, ice lenses tend to be oriented perpendicular to the heat flow, i.e., parallel to the culvert walls as discussed in 2.2.1.1.

3.1.1.2 During Thawing

1. Thawing commences, usually, from the culvert walls towards the backfill soil. Upon thawing, the water released from the ice lenses oriented around the culvert will cause local over-saturation. This is because the released water is trapped between the culvert walls and the still frozen backfill.
2. This case of super-saturation exists until the water is drained, i.e., until the soil has completely thawed.

3.1.2 Structural Analysis

If frost action occurs on the soil around the pipe arch, ice lenses will be oriented parallel to the wall developing heaving pressure perpendicular to the wall. Let us examine the effect of heaving pressure on the various plates of the pipe arch structures.

3.1.2.1 Numerical Example

Consider a pipe arch conduit with a horizontal span (S) and a rise (H) of 3.12 and 2.05 m respectively, as shown in Fig.3.2. Assuming a 1.2 m depth of soil cover (D) and a soil unit weight γ of 17.2 kN/m³. According to the Ring Compression Theory the soil pressures P at any point around the conduit are shown in Fig.3.2. as computed by the following equation.

$$P = \frac{\gamma DS}{2R} \quad (3.1)$$

where R is the radius of curvature at the point under consideration.

Herein, let us assume two kinds of soil backfill as follows:

1. Soil A is a well graded soil with 3% by weight finer than 0.02 mm. According to the gradation characteristics, D_{10} as the effective diameter would be about 0.045 mm. Thus, a heaving pressure of 20 kPa (2.9 psi) is obtained from equation 2.5.
2. Soil B with 15% by weight finer than 0.02 mm, and a D_{10} value of 0.011 mm. Thus the heaving pressure is 84.0 kPa (12.15 psi).

Therefore, the behavior of the bottom and the corner plates could be analyzed as follows:

The Bottom Plate:

It is assumed that the soil pressure outlined in Fig. 3.2 does not cause bending moment to the bottom plate. Yet, the soil pressure at the bottom plate is 7.2 kPa, i.e., less

than the heaving pressure of both soils A and B. Therefore, the bottom plate will be subjected to bending moment during freezing period. Moreover, the soil underneath the conduit may heave towards the center of the conduit, uplifting the bottom plate which causes decrease in the vertical dimension and increase in the horizontal dimension and at the same time providing more space for ice lenses to form within this soil which eventually may cause over-saturation during thawing.

The Corner Plate:

Soil A: The soil pressure at the corner plate is 70.45 kPa, i.e., larger than the heaving pressure (20 kPa). Therefore, it is expected that the ice lenses cease to grow, i.e., the soil at the corner will not be subjected to frost action. However, at point "A" just above the joint, this condition could be reversed and an inward displacement may occur at this point.

Soil B: In case of soil B the heaving pressure is 84 kPa which is larger than the soil pressure at the corner plate. Therefore, the soil may heave towards the corner plate causing moment and displacement in the corner plate and also providing more space for ice lenses to form.

Moreover, the ice lenses may grow in the direction of the native soil. This may cause displacement towards the native soil which could be low strength clay or silt which deforms permanently under pressure. Thus after thawing, the density of the backfill will be reduced, i.e., the required support is lost and the plate pressure is not relieved. Subsequently, the pressure must be taken by the corner plate itself. If this pressure becomes greater than the plate can bear, the plate fails. This failure is characterized by horizontal cracking through valley bolt holes at the top corner plate and by crimps at the haunch. Furthermore, the imposed load will be supported by the top plate instead of the

corner plates. Consequently, a large deflection at the crown will occur. Also, a condition of loss soil support may develop during thawing.

3.2 Laboratory Model

As outlined above the main problem in pipe arches could be the loss of soil support at the coroners. Therefore, a laboratory experiment was performed to study this effect on the structure. The model was designed to simulate a corrugated steel pipe arch culvert, and thus the induced strains and deflections due to the loss of soil support could be measured. The major components of the laboratory model are described as following:

3.2.1 Soil Container

A soil container, 2.70 m by 0.95 m by 1.25 m high, was built of 20.0 mm wood sheets supported by 75 x 50 x 5 mm steel angles at the corners. Only one side of the container was made of 25.0 mm Plexiglas so the culvert could be seen during testing as shown in Fig.3.11.

3.2.2 Soil

A uniform clean dry sand was used in the experiment. The geotechnical properties of the soil are described in Appendix A.

3.2.3 Culvert

A pipe arch culvert made of corrugated aluminum sheets was shaped and manufactured by the Central Research Shop of the University of Windsor. The 1 mm thick aluminum sheets were corrugated to the dimensions shown in Fig. 3.3. Also, the culvert dimensions are shown in Fig. 3.4. The mechanical properties of the aluminum and the cross section properties are described in Appendix A.

3.2.4 Wooden Dowels

Six wooden dowels 25.0 mm diameter by 1.20 m length were placed, during compaction, behind each corner plate. The dowels were passed through holes in the back wood side parallel to the conduit. Roughly 0.25 m of each dowel was left outside the container so it could be pulled out of the container. Fig.3.5. shows the location of each dowel behind the corner plate.

3.2.5 Hydraulic Ram

Load was applied with a 70 kN capacity hydraulic ram. The magnitude of the load was measured with a calibrated strain-sensing load cell and a strain indicator. The applied load was uniformly distributed through a 100 mm depth steel I beam placed on 0.20 m width by 0.95 m length wood panel. The whole assemble was placed on the top of the soil and centered with the culvert crown as shown in Fig. 3.5.

3.2.6 Instrumentation

Twelve different locations on the mid-length of the culvert were chosen to install 24 strain gauges as shown in Fig. 3.6.a and 3.6.b. At each location, one strain gauge was placed on the valley and the other on the ridge as shown in Fig.3.7. Six dial gauges were mounted on the same cross section. All the strain and dial gauges were zeroed before replacing any soil around the culvert. The output of the gauges was recorded during construction (after compacting of each layer) and after applying each load. Fig. 3.8. shows the experiment set-up before the backfilling operation.

3.2.7 Compaction Procedures

A 200 mm thick sand layer was compacted on the bottom of the soil container. Then a 50 mm thick layer of sand was pre-shaped to the invert radius of the culvert without compaction to allow structure relaxation into bedding as shown in Fig. 3.9. Sand was then placed in 100 mm thick layers and compacted on each side of the conduit. Meanwhile, wooden dowels were placed in the desired location as shown in Fig. 3.10. After the sand reached the crown of the culvert, three 75 mm thick layers were added on the top of the crown. Compaction was performed by using an electrical plate vibrator for two minutes on each layer. Fig. 3.11 shows the model after completion of construction and before applying loads.

3.2.8 Experimental Program

After compacting the soil on the top of the culvert, a 200 mm wood panel was centered with the culvert and load was applied at the center of the panel to assure uniform load distribution. The load was applied first with increments of 4.45 kN up to 13.35 kN, without disturbing the soil behind the corner plates. Then, the dowels were pulled subsequently out of the soil container leaving their voids to be refilled by the surrounding soil. Thus, the degree of the compaction of that soil is reduced or in another words the bearing capacity of the soil was reduced. The dial and strain gages output were taken before and after pulling each dowel. As the dowels were being pulled out, the deflection at the crown was increasing. When the last two dowels were pulled, the deflection at the crown increased excessively until the failure, as shown in Fig. 3.12. and 3.13.

3.2.9 Test Results

The strain readings were interpreted to calculate moments and thrusts for each loading condition. Figures 3.16 to 3.21 show the bending moment diagrams, axial force diagrams, and the deflections of the conduit during testing. The following conclusions may be drawn out of this test:

1. In soil-steel structures it is usual to ignore bending moments and design the metallic shell for thrust only. However, the results of this test show that the moment at the corner plate increased from 21.24 to 101.06 N.mm when three dowels were pulled from each side. In other words, excessive bending results from the loss of soil support which could be attributed to frost action.
2. As the soil was losing its support behind the corner plate and the moment was increasing at that plate, the moment at the crown was reversed, i.e., changed from 50.9 to -395.7 N.mm till plastic hinges were developed at and near the crown as shown in Fig. 3.16 and 3.17. When these hinges were formed the moment at the corner plates did not increase any more. Instead the moment and the deflection increased excessively at the crown till the failure, as shown in Fig. 3.14 and 3.15.
3. Although the Ring Compression Theory assumes that the circumferential thrust is uniform around the conduit wall, the test results show a non-uniform thrust distribution exists even when the load is concentric with the maximum value near the crown as shown in Fig. 3.18 and 3.19. Moreover, when the soil lost its support, the bottom and corner plates were subjected to tension forces.
4. The results of the dial gauges (#1,#4) and (#3,#5) show that before pulling the dowels and with applying load 13 kN the decrease in the vertical dimension was

2.0 mm and the increase in the horizontal dimension was 2.1 mm. At the same load when the dowels were pulled the former was 15 mm and the latter was 12 mm as shown in Fig. 3.20 and 3.21.

5. Bolt tearing did not occur in the model because they were not simulated; instead the plates were riveted.

It is of a great interest herein to see that the distress observed in the model were similar to that reported by the MTO. Therefore, it could be concluded that the loss of soil support causes distresses in soil-steel structures and any thing resulting in loss of soil support must be prevented.

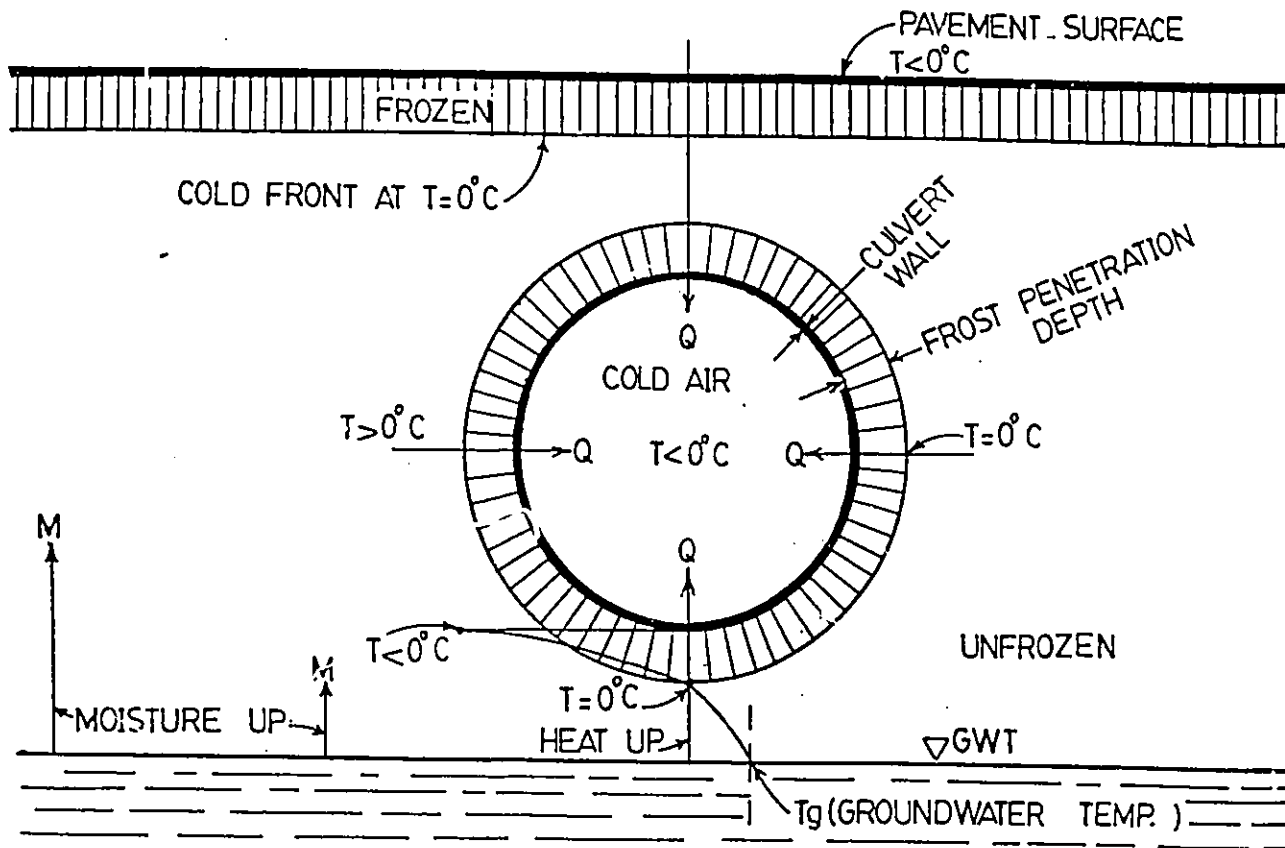


Figure 3.1: Temperature profile of the soil-culvert interaction.

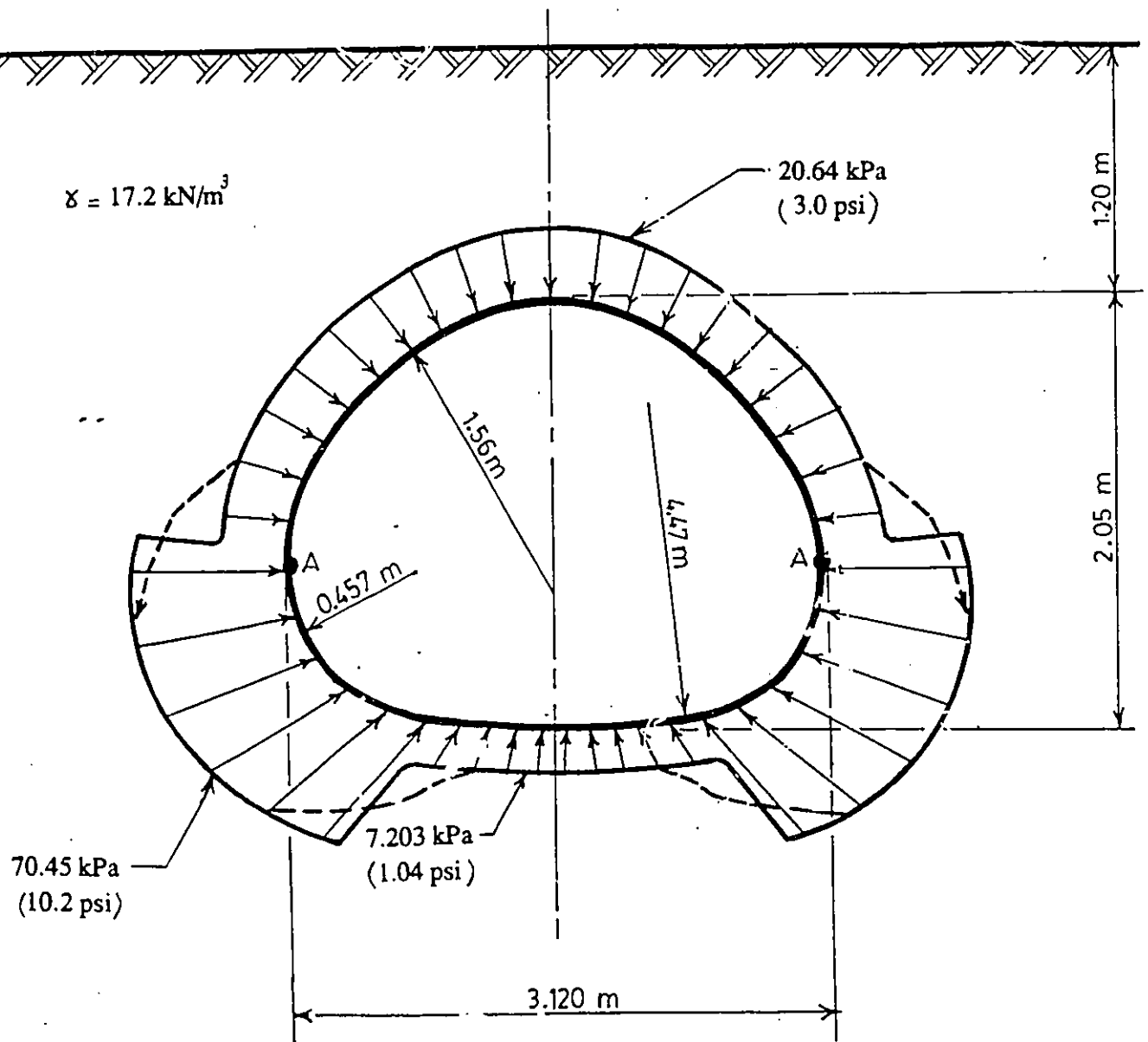


Figure 3.2: Effect of heaving pressure on pipe-arch conduit (numerical example)

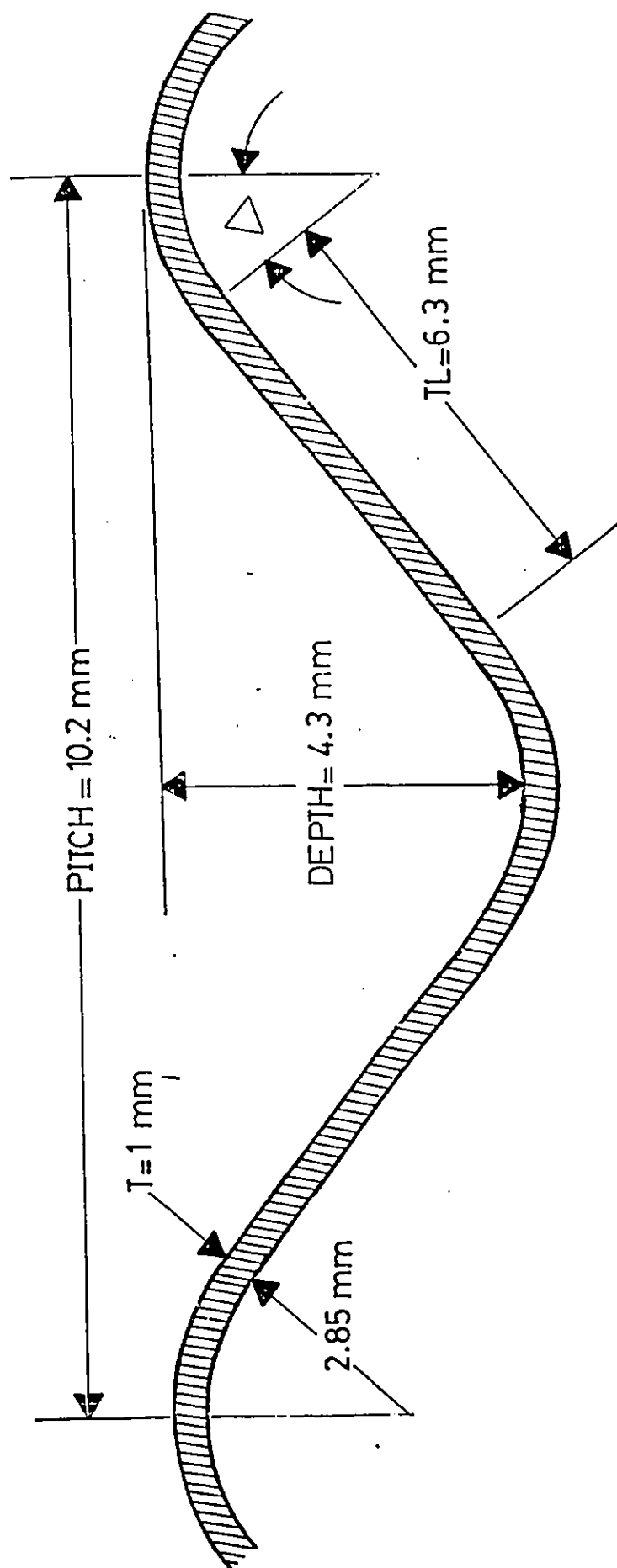
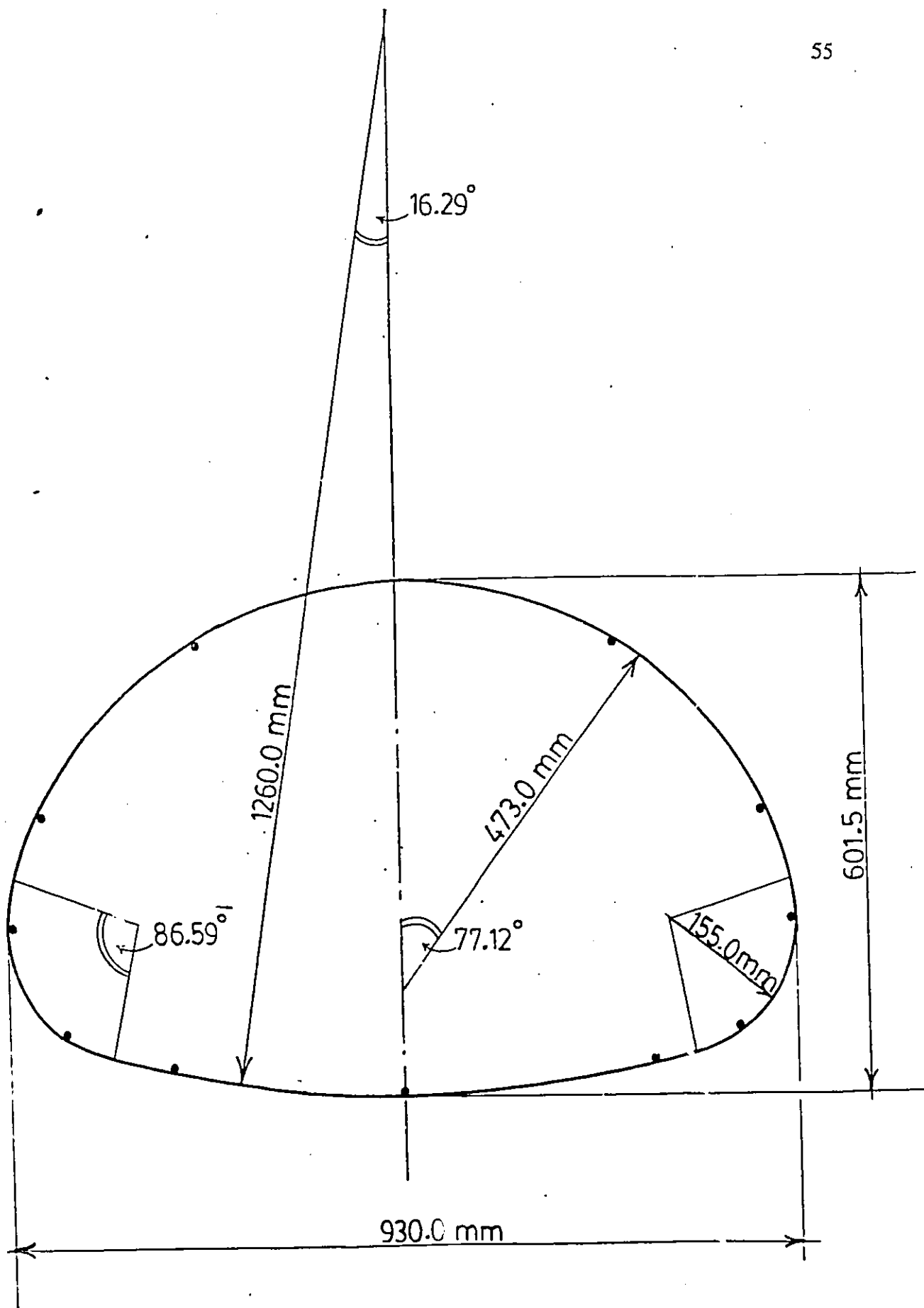


Figure 3.3: Corrugation profile of the conduit used in the laboratory model.



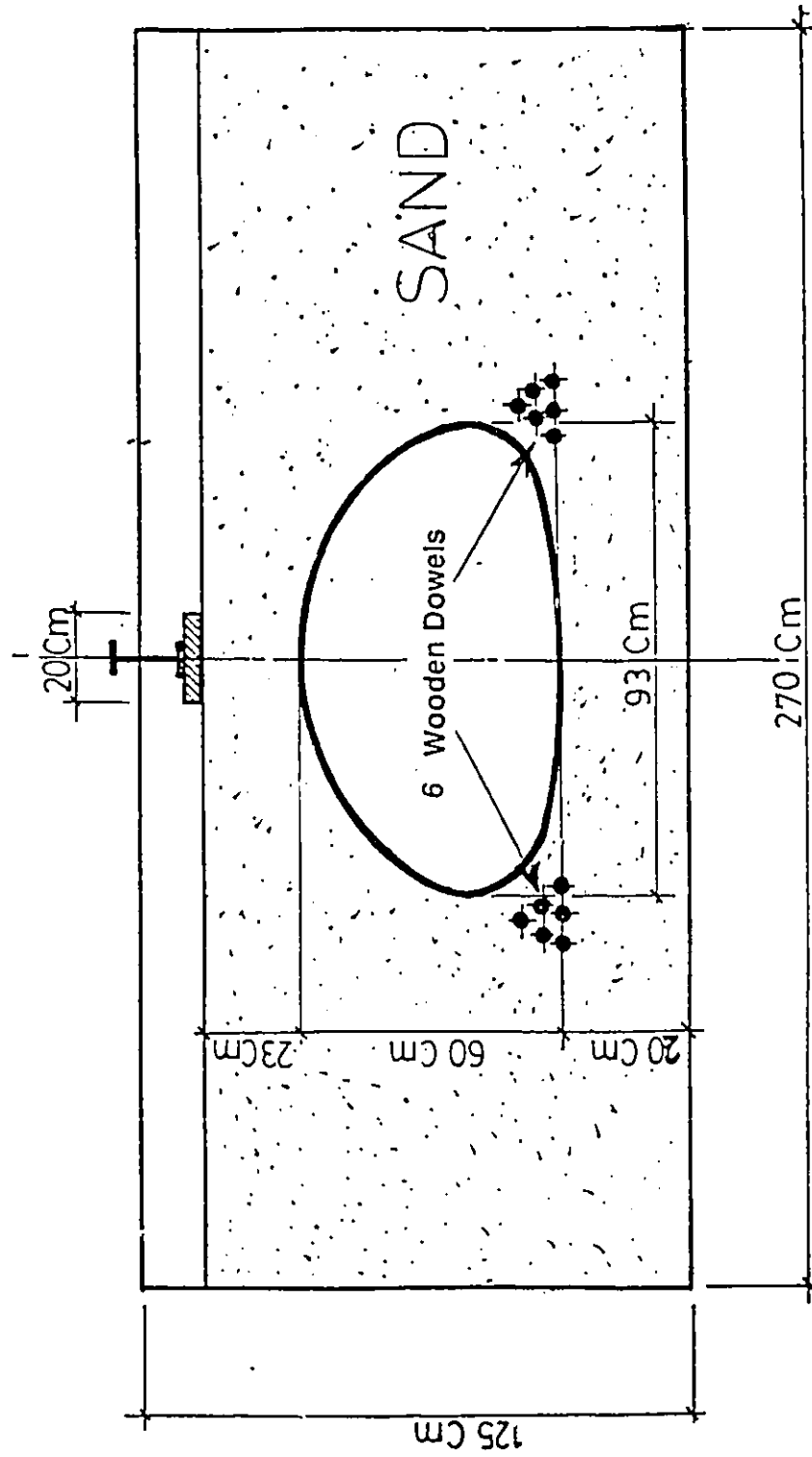
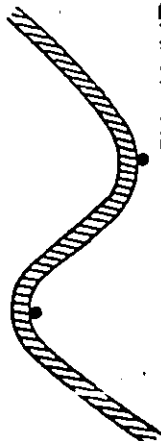


Figure 3.5: Detailed drawing of the experimental model

ODD NUMBERS
(1, 3, 5, 7 → 23)



EVEN NUMBERS
(2, 4, 6, 8 → 24)

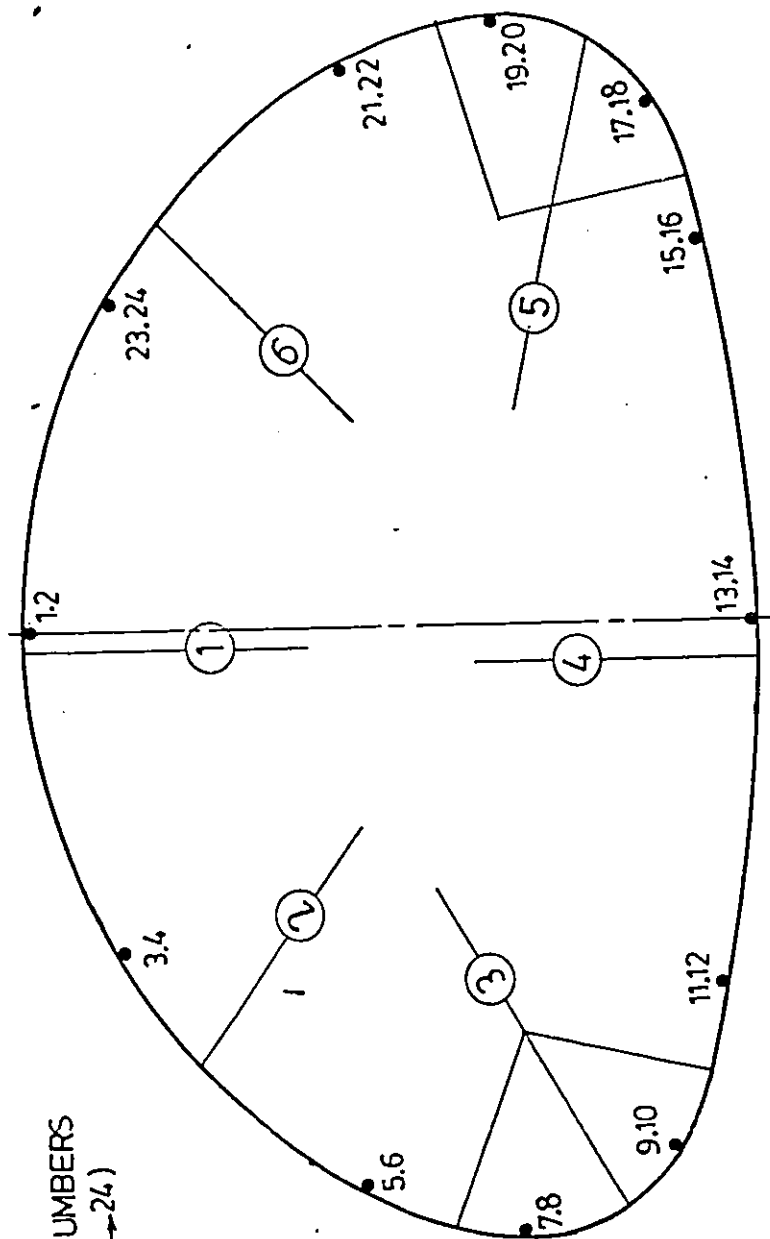


Figure 3.6: Cross section at the mid-length of the culvert showing the location of 24 strain gauges and 6 dial gauges.



Figure 3.7: The conduit before placement in the container.



Figure 3.8: The experimental set-up before the backfilling operation.



Figure 3.9: The Bedding Shape Before Placing The Conduit.

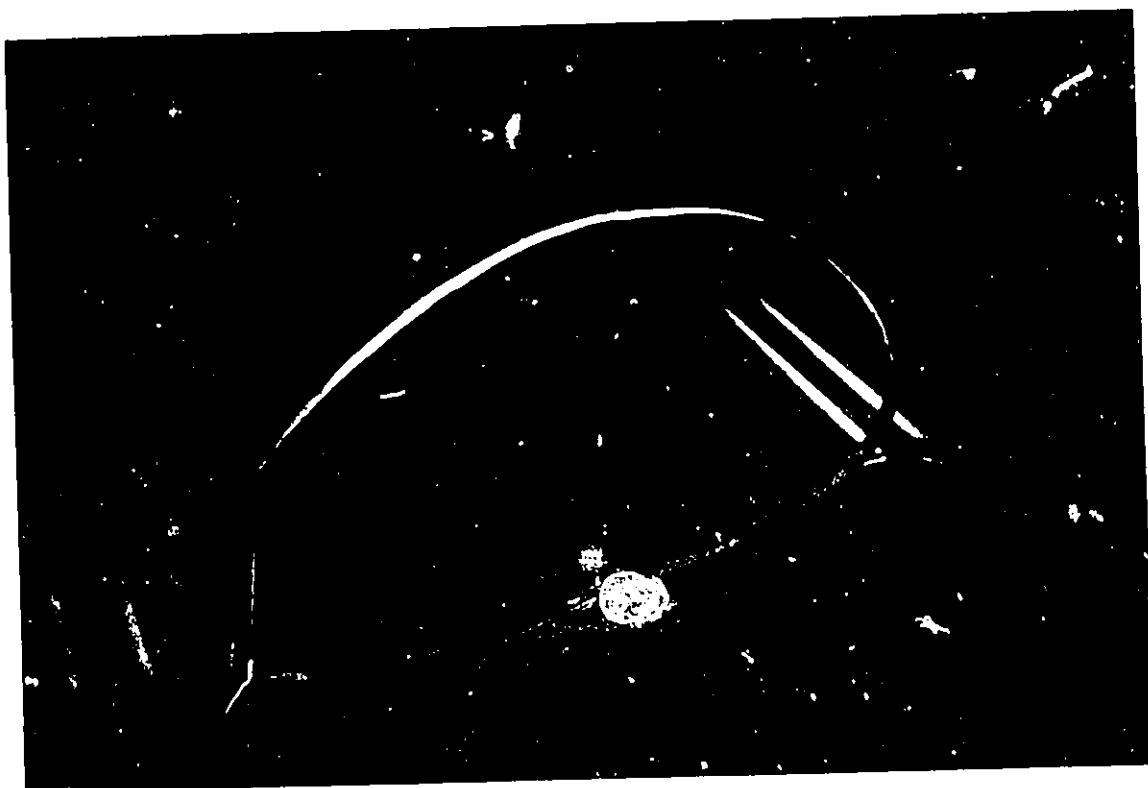


Figure 3.10: Placement of the wooden dowels through the soil.



Figure 3.11: The experimental set-up before applying the loads.

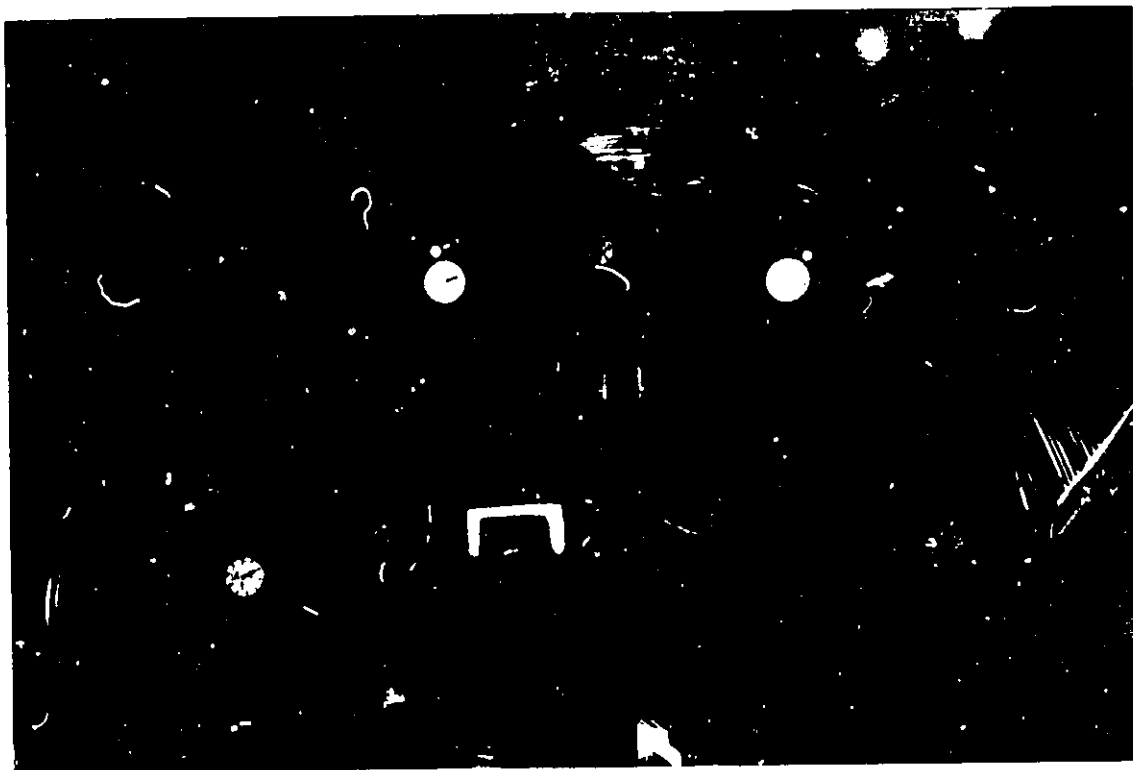


Figure 3.12: Beginning of crown deflection at a load of 13 KN and after pulling three dowels from each side.

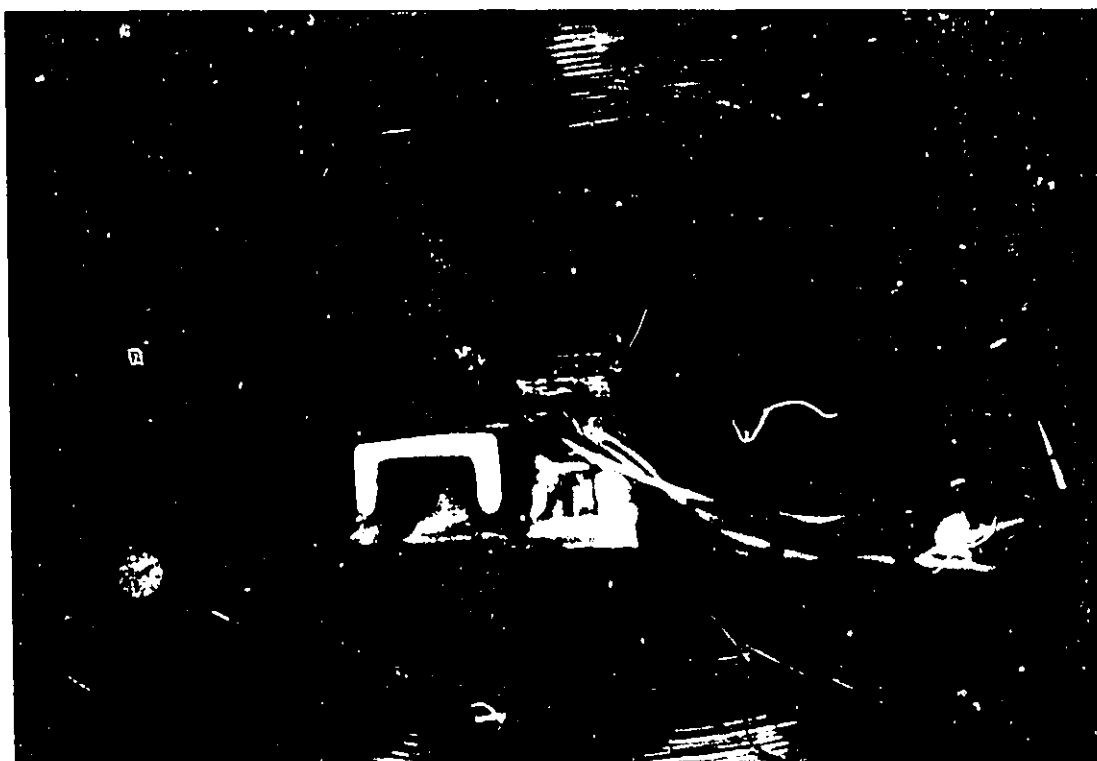


Figure 3.13: Formation of plastic hinges at crown and shoulders at a load of 13 kN and after pulling 5 dowels from each side.

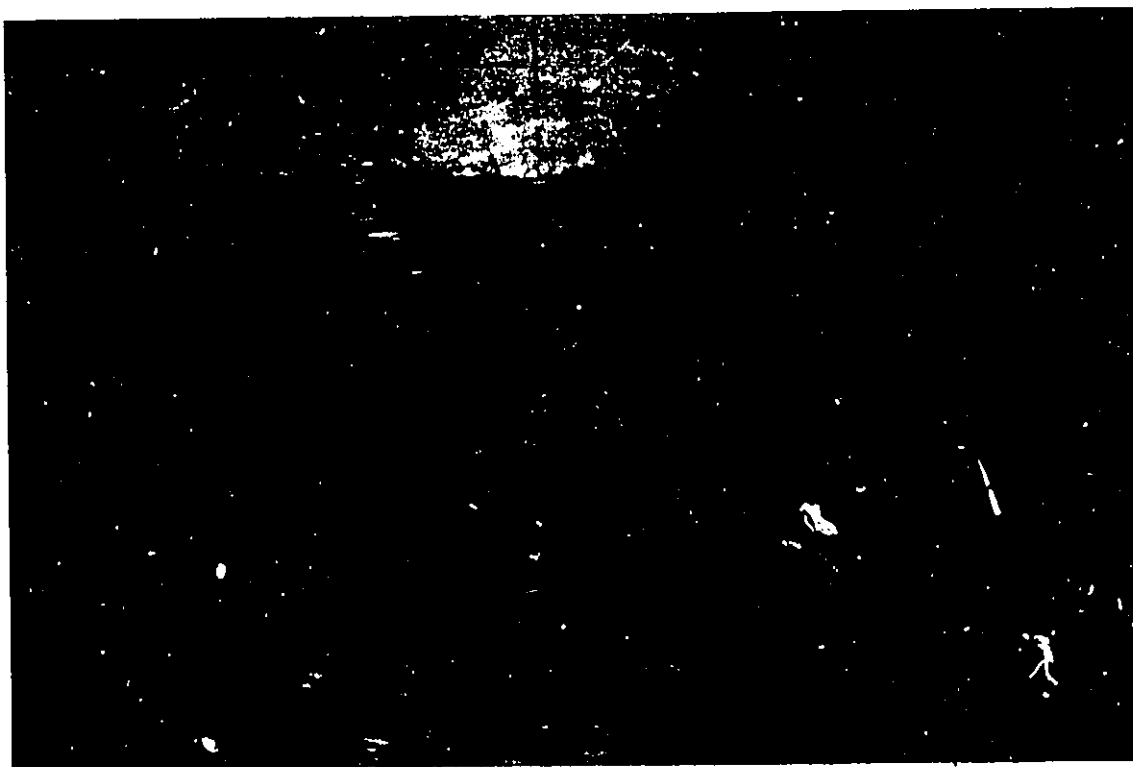


Figure 3.14: The Conduit Shape at a Load of 8 kN After Pulling All The Dowels.

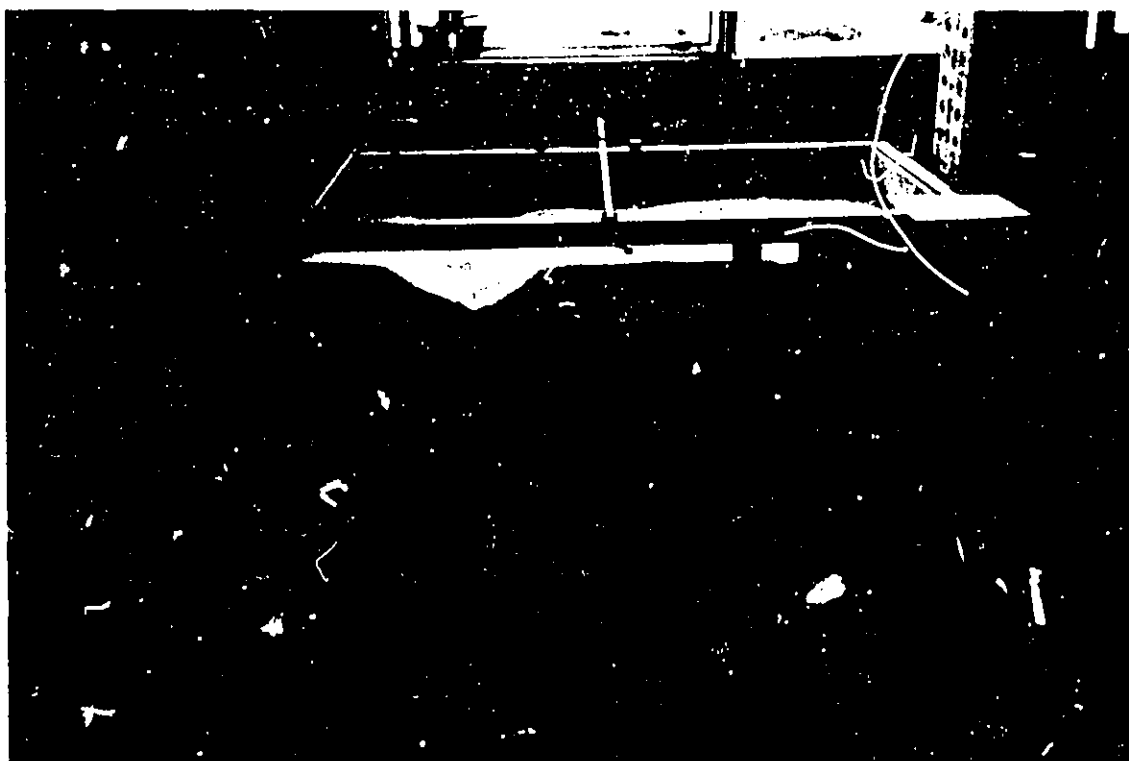


Figure 3.15: Failure at the conduit and at the top soil after unloading

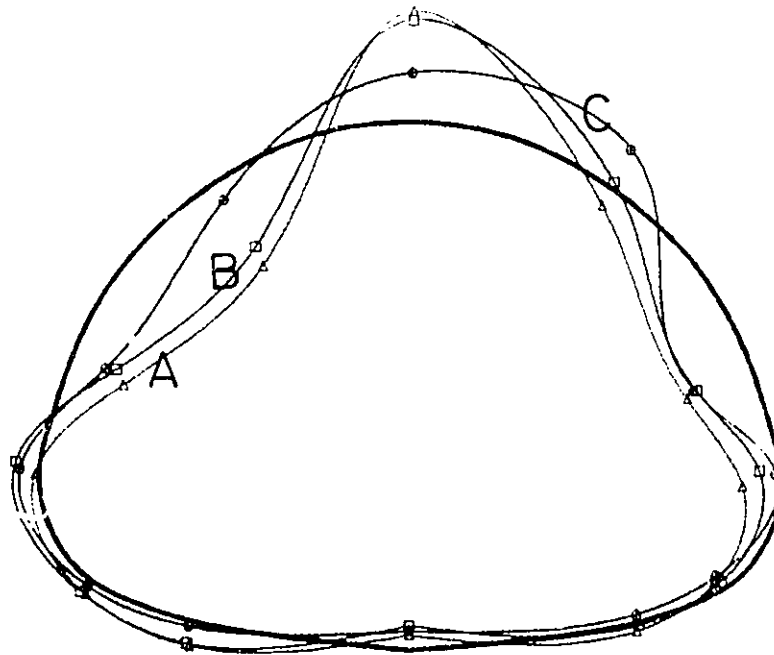


Figure 3.16: The bending moment diagram before pulling the dowels for three load conditions (A) Soil at crown; (B) Soil at 23 cm above the crown; and (C) Load of 13 kN.

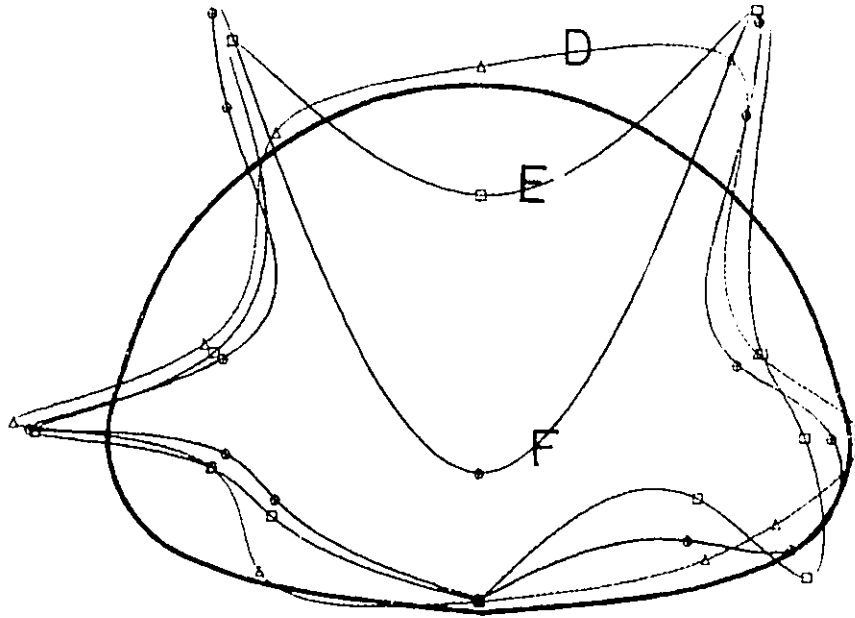


Figure 3.17: The bending moment diagram at a load of 13 kN, for three soil support conditions: (D) After pulling three dowels from each side; (E) After pulling five dowels from each side; and (F) After pulling six dowels from each side

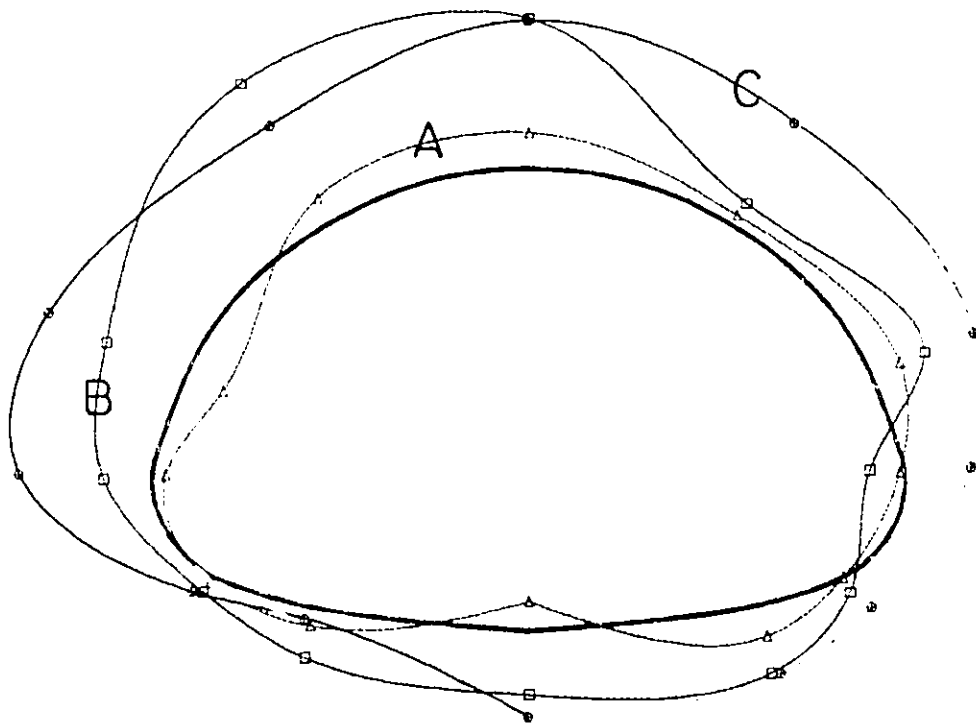


Figure 3.18: The axial force diagram (thrust) before pulling the dowels, for three load conditions: (A) Soil at crown; (B) Soil at 23 cm above the crown; and (C) Load of 13 kN.

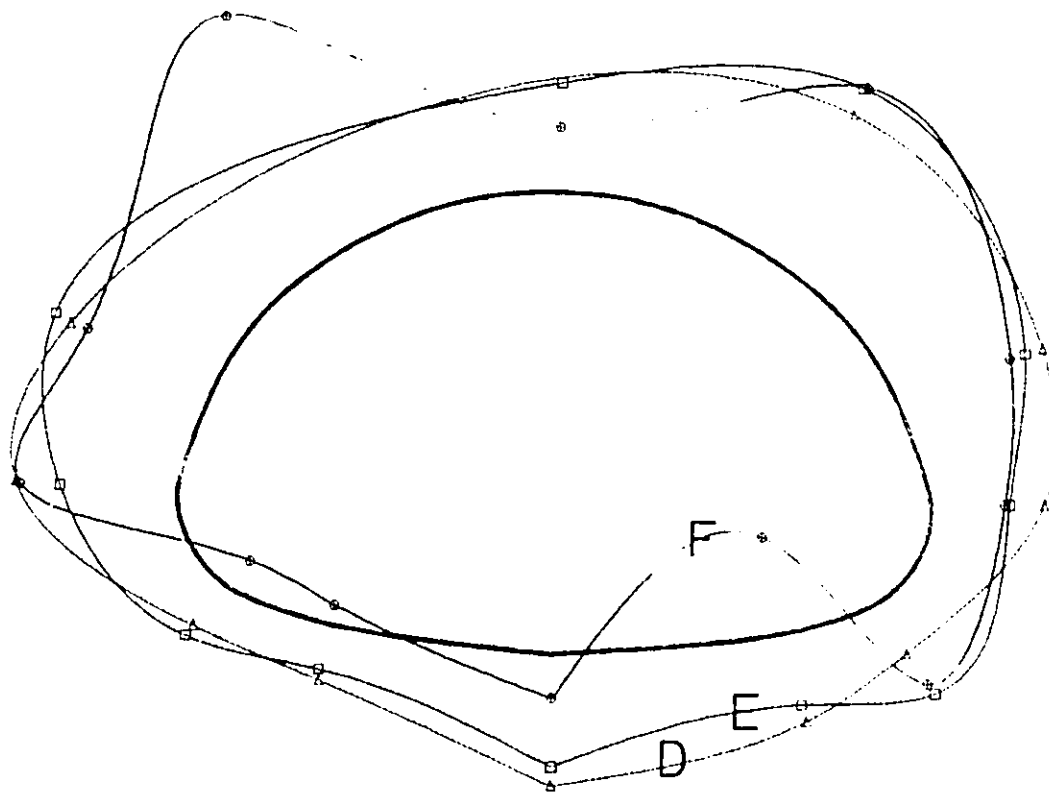


Figure 3.19: The axial force diagram (thrust) at a load of 13 kN, for three soil support conditions: (D) After pulling three dowels from each side; (E) After pulling five dowels from each side; and (F) After pulling six dowels from each side

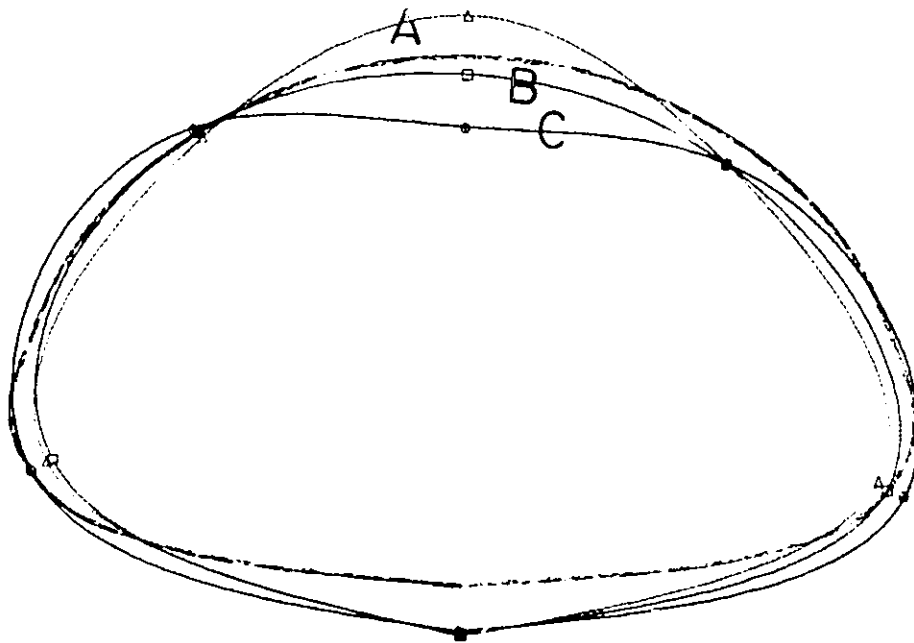


Figure 3.20: The deflection before pulling the dowels, for three load conditions: (A) Soil at crown; (B) Soil at 23 cm above the crown; and (C) Load of 13 kN.

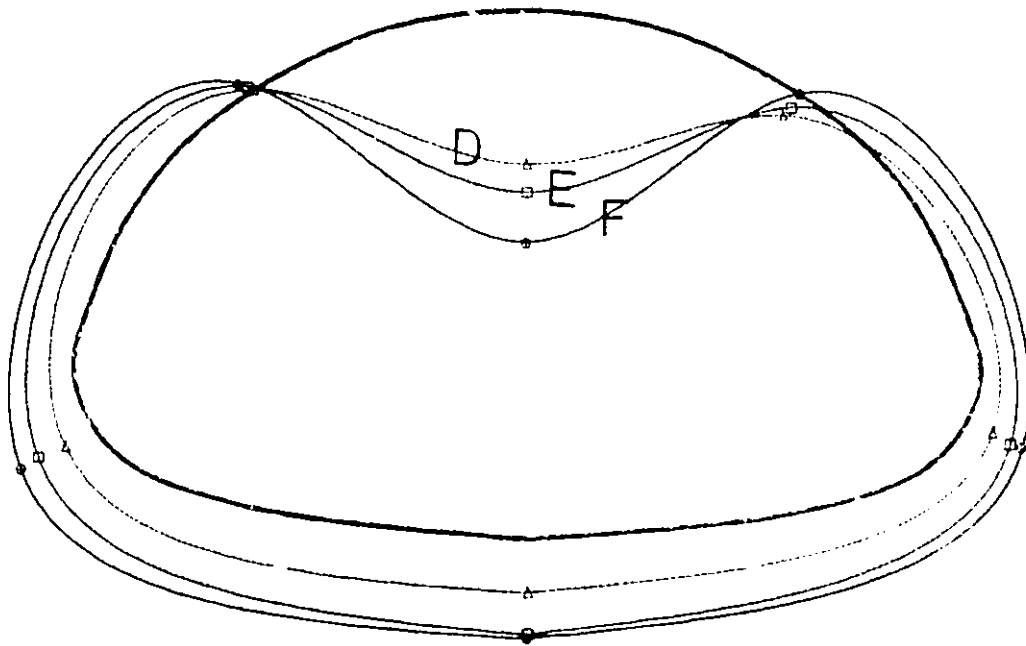


Figure 3.21: The deflection at a load of 13 kN, for three soil support conditions (D) After pulling three dowels from each side; (E) After pulling five dowels from each side; and (F) After pulling six dowels from each side

Chapter IV

TREATMENT

4.1 General

There are three conditions necessary for frost heave and formation of ice lenses to occur: (1) temperature must be sufficiently cold and prolonged that the soil water freezes, (2) the water table must be close enough to the freezing front in the soil mass so that water can migrate to a growing ice lens, and (3) the soil must be susceptible to the formation of segregated ice. The basic approach to prevent frost heave and/or control the loss of bearing capacity of the soil is to eliminate one or more of these conditions. Within the usual economic limitations there is very little which can be done to control the two former factors. However, in highway engineering practice, the remedy is to lower the ground water table to a sufficient depth so water cannot be sucked up to the freezing front. Yet, this remedy does not suit all kinds of soil-steel structures since most of soil-steel structures are used as culverts, and thus the soil will be saturated even if the ground water table was low. The other option is to use a non frost susceptible backfill. Yet in many areas, the supply of clean granular soil is expensive if not scarce. Also, washing the fines out of the dirty granular soil is a difficult and expensive operation. Therefore, the available soil on site must be treated. The last few decades have seen many attempts to inhibit the problem of the frost heave and the consequent results occurring in the highways and roads. The most common methods which could be used in the case of culverts are outlined in this chapter.

4.2 Remedies For Frost Action

4.2.1 Chemical Additives

In the early fifties, the use of chemical additives was suggested [76], [41] as a treatment to lower the freezing temperature of soil water. It was found, for example, that the presence of 10% of Calcium Chloride or Sodium Chloride in the soil lowered the freezing point of pure water from 0 to -6° C. The freezing point lowering is directly proportional to the amount of chemical present. Therefore, lower temperature is required to produce ice lenses that cause the frost heave. Also, there are two promising additives [8], Calcium Lignosulphonate and Sodium Tripolyphosphate. They have a marked effect in reducing water absorption. Various concentrations of these two materials were mixed with water and added to silty clay soils. Both chemicals were effective in reducing frost heave but was proved too costly [12]. The addition of only 0.5% of that tripolyphosphate by dry unit weight of soil reduced the frost heave of a highly frost-susceptible soil to a value very close to zero and a concentration of 1% prevented the heave completely.

There are two disadvantages to the use of these chemicals with soil-steel structures. Firstly, the maximum life of such treatment is 3 to 5 years. Yong and Janiga(1971) [89] have shown that while water moved into the freezing zone, salts moved away from the freezing zone toward the warmer temperature. Secondly, they could have detrimental effect on the conduit material.

4.2.2 Thermal insulation

Thermal insulation has been successfully used in construction to attenuate freezing and thawing of the ground. Its main function is to reduce the loss of ground heat to the air. Numerous insulated road test sections have been constructed in Canada and the

northern regions of the United States since the early sixties, [14]. The results were used as basis for developing a relationship between the freezing index and the required thickness of insulation. As an example, a 50 mm thickness of expanded polystyrene insulation reduced the frost penetration at Sudbury by 0.9 m during the winter (freezing index 1450 Celsius degree days). In 1966, The Ontario Ministry of Transportation began using expanded polystyrene to insulate the subgrade at frost heave sites. The Ministry also used expanded polystyrene to insulate sewers and water mains to prevent freezing of water inside the pipes. The material is manufactured by Dow Chemical Canada in the form of boards 1.20 x 0.60 m with thickness of 25, 50, or 75 mm under trade name of "STYROFOAM*HI". It possesses excellent water resistance and a very high compressive strength in addition to a very low thermal conductivity. One disadvantage of this insulation when used under road pavement is the possibility of ice formation on the pavement surface because of the retardation of the upward flow of heat from the ground towards the ground surface. This means that the surface of such pavement must be spread with salt or some kind of de-icing chemicals to minimize traffic hazards. Thermal insulation has been successfully used in Japan and Europe under railway tracks, behind retaining walls and around tunnels.

The use of thermal insulation around soil-steel structure is a very tempting idea. The insulation could be designed in such a way that the supporting soil would never freeze (using the Freezing Index value). Also, the formation of ice on the inside walls of the culvert does not matter, because here icing does not impair traffic safety as in the case of highways. In addition, the material is inexpensive and has already been successfully used in many other structures. Jumikis (1973) [33], recommended the use of thermal insulation around the outside walls of the rigid culvert to reduce the "soupy" soil

conditions around the culvert upon thawing. Herein, the insulation would prevent the trapping of the moisture between the still frozen soil ring and the culvert's wall and the subsequent super-saturation of the soil during thawing.

In the case of soil steel structure, it is assumed that there is no independent movement of the backfill after completion of the construction, i.e., the flexible steel conduit must develop composite action with the surrounding polystyrene soil. Thus, the insulation boards must have the same flexibility so they act with the corrugated sheets and the soil as one section. This insulation material must also have a compressive strength larger than the soil pressure. The friction between the soil and the insulation boards must be taken into consideration. Further research is required to study the effect of insulation boards on the flexible conduits.

4.2.3 Portland Cement

Portland cement can considerably reduce or even prevent the frost heave of soils containing appreciable quantities of fines, when mixed with soil and enough water for hydration [35]. The cement works to reduce permeability, plug the pores, and to bond the grains together. Thus, it prevents the formation of small ice lenses. The non-heaving characteristic of the concrete is a good example for that. The soil material in the soil-cement mixtures [36] can be almost any combination of gravel, crushed stone, sand, silt, and clay. The Portland Cement Association [58] has set two kinds of soil cement (1) cement-treated soil and (2) cement-modified soil.

4.2.3.1 Cement-Treated Soil

This soil contains 5 to 10% cement by weight [55]. This amount of cement is usually added to dirty granular soil to:

1. provide uniform, strong, firm inexpensive material with high support value during the different climatic conditions.
2. resist frost action.
3. prevent moisture movement.
4. resist intrusion of the fine materials in the underlying soil into the supporting material.
5. reduce or prevent volume change of the soil.

These properties seem to be adequate for soil-steel structures since they remain the same after many cycles of freezing and thawing. Yet the treated material is not soil any more but a hardened rigid material. Granular material in AASHTO Soil Classification Groups A-1, A-3, A-2-4, and A-2-5 have the most favorable characteristics and generally require the least amount of cement for adequate hardening. The PCA has done extensive research to determine the required cement content for the various AASHTO soil groups. It was found that the percentage of cement required is different from one soil to another with a minimum value of 5% for soil A-1-a. Cement-treated soil can be used with the soil-steel structures only if it is considered in the design as a solid material around the culvert.

4.2.3.2 Cement-Modified Soil

This is the soil material that has been treated with a relatively small amount of Portland Cement (1% to 4% by weight) less than that is required to produce hardened soil-cement. With the small quantities of cement generally used, cement-modified soil becomes caked or slightly hardened [57]. The 28-day compressive strength is about 0.5 MPa (75 psi). Hence, it still functions essentially as a soil, though an improved one. This mixture is used when the supply of acceptable granular material is a problem or is too costly. The primary objects of the cement-modified soil are to:

1. increase the bearing capacity and the consistency of the soil providing firm and stable support which resist local settlement.
2. reduce or eliminate the plasticity index.
3. increase the resistance to the moisture movement.

The Portland Cement Association [57] ran test on a cement-modified material from California. The soil was classified as A-1-b. Table 4.1 shows that the addition of 2% cement by weight increased the value of the CBR from 43 to 255. The addition of 4% cement increased the value to 485. Also, after 60 cycles of laboratory freezing and thawing the CBR value was about the same for the 2% and increased to 574 in case of 4%. Also, the Portland Cement Association conducted an outdoor test on cement-modified granular soils. It was found that after five years of exposure to freezing and thawing, even with a cement content as low as 1.5% by weight, the treated material was still able to support considerably more load than the untreated material. Cement-modified granular soils are being used extensively as pavement bases for highways and streets when the available soil requires only a small degree of improvement.

The proper water content for cement-modified soil and cement-treated soil is determined [58] by the moisture density test. This test determines the relationship between moisture content of the soil-cement mixture and the resulting density when the mixture is compacted before cement hydration with a standard compacting force. Thus, the maximum density and the optimum moisture content could be determined. The three components are mixed in place using travelling mixing machine or in a central mixing plant. Then, the material is spread and compacted to the maximum density. After completion, the material could be cured by waterproof paper, plastic sheets, cotton mats, wet burlap, or fog-type water spray. In case of cement-modified soil curing may be omitted, although a cure coat will result in maximum benefit from the cement.

4.3 Frost Action Effect On Cement-Modified Soil

The use of a small percentage of cement as a soil treatment could be much cheaper than washing or replacing the available soil. It provides a more durable and stable support during spring thaw. Furthermore, the use of this modified material could reduce the required thickness of the conduit wall improving the structural properties of the soil. Hence, the frost action effect on cement-modified soil must be tested, so that the use of this treated soil around the conduits could be evaluated. The literature published by the PCA in Canada and U.S.A. related to the cement-modified soils, and that by the U.S. Corps of Engineers (Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire) indicated no frost action test was ever done on an open system of cement-modified soil. Therefore, the effect was studied by the author by two means as follows:

1. analysing the size of the soil grains with and without treatment, and thus observing the changes in the structure of the particles.

2. experimenting the behaviour of the treated soil under a unidirectional freezing, i.e., observing both the frost heave and the increase in water content for soils with and without treatment.

4.3.1 Soil Samples

Three pre-fabricated soils A, B, and C were used to examine the effect of frost action on them when mixed with cement paste. The grain size distribution curves for the three soils are given in Fig. 4.1. and could be described as following:

1. Soil A was a mixture of 80% sand and 20% silt. From the curve it could be inferred that soil A is a poorly graded soil with 13% of soil finer than 0.02 mm. According to Casagrande's criterion this type of soil is frost susceptible soil since it has more than 10% material finer than 0.02 mm.
2. The mixture of soil B was designed to represent a better graded soil and with less fine materials than soil A. Therefore it was a mixture of 87% coarser sand than the above one and 13% silt having about 8% finer than 0.02 mm.
3. Soil C was designed to represent a less frost susceptibility soil than the other two soils and at the same time posses a percentage of materials finer than 0.02 more than 3%. Therefore it was a mixture of 91% medium sand with 9% silt and clay having about 6% finer than 0.02 mm.

For each of the three soils, the main components (sand and silt) were oven dried for 24 hours and then mixed for 15 minutes in an electrical mixer to obtain a homogeneous soil sample. A summary of the soils properties and classification is given in Table 4.2.

4.3.2 Experimental Program

Each soil was first tested without cement; then two different percentages of cement were chosen for each soil. The cement was always added in the form of paste. The percentage of cement added to the soil was chosen to eliminate or reduce the frost heave considerably. Hence, soil A was tested with 3% and 5%, soil B with 1.5 and 3% and soil C with 1.5% and 3% cement by dry weight of soil. For each of the nine soils two experiments were run: (1) grain size analysis, and (2) frost action test

4.3.3 Grain Size Analysis Test

The sample preparation for this test was done by mixing soil with a paste of cement for about 15 minutes in an electrical mixer, then keeping the mix in a sealed plastic bag for 72 hours for hydration. A 500 g sample was taken from each soil and oven dried for 24 hours then sieved for about 20 minutes in an electrical shaker. The grain size distribution curves for the nine soils are presented in three diagrams as shown in figures 4.2, 4.3, and 4.4 for the soils A, B, and C respectively. The results show that the cement works to bond the small particles together and thus, decreasing the percentage of the fine materials and converting the frost susceptible material to a less or non-frost susceptible soil. Herein, it must be recognized that if that soil was compacted, the cement will also work to plug the pores and to reduce capillarity.

4.3.4 Frost Action Test

4.3.4.1 Description of Equipment

Over the past 40 years, many frost cells were designed to simulate the unidirectional penetration of freezing under a temperature gradient to the ground and thus to measure the heave and/or the heave pressure. These cells were designed by the U.S. Corps of Engineers (CRREL), National Research Council, or researchers at some universities in North America to study a particular problem at hand. After contacting these institutions, the author found that none of these frost cells was available to be purchased or rented. Therefore, a simple frost cell was designed and fabricated by the author at the University of Windsor to study the frost action effect on cement-modified soil. Hence, it was very important in the design of the cell and the experimental program to keep all the parameters (water content, degree of saturation, dry unit weight, temperature gradient, and freezing temperature) constant for each soil group and thus the results of the soils in the same group could be compared and the effect of cement could be evaluated. It is important here to mention that the results of this frost cell can not be compared with any of those of the other cells since the parameters (temperature gradient and freezing temperature) are different.

The frost cell was designed to hold a sample of soil 140 mm in diameter and 140 mm long, as shown in Fig 4.5. A mold of the required size was made of 70 mm thick polystyrene to prevent radial heat flow. To initiate freezing from the top only and to prevent the downward heat flow, the base was made of a 40 mm thick PVC plate. To start the freezing process, the whole cell was moved to a freezer and temperature was maintained at approximately -4°C . As the sample heaved, the change in height was measured by sensitive callipers. A vertical pressure of 3.4 kPa (0.5 psi) was applied by

placing a dead weight of 48 KN (10.8 lb) on the top of the sample to simulate the overburden condition. To simulate an open system with the ground water table, a horizontal pipe was connected to the bottom of the base plate with the opposite end attached to a 50 mm diameter burette filled with water. A valve was placed in between the burette and the base plate to regulate the flow of water before the sample was placed. The water level at the burette was kept constant at mid-depth of the sample during freezing time. To keep the water temperature in the burette and in the pipe from freezing all the time during freezing process, each of the burette and the pipe was wrapped with an electrical heating cable. The cable was connected to a thermostat to turn the heat on when exposed to temperature below 3° C the temperatures along the length of the specimen was monitored with three thermistors placed at the center of the sample at equal distance (i.e., at the top, mid-depth, and the bottom) as shown in Fig.4.6. Appendix B lists equipment manufactures and their specification.

4.3.5 Sample Preparation

4.3.5.1 Samples without cement

Each soil sample first tested without adding cement to measure the initial heave. Sample preparation was started by filling the burette with water to the level of the bottom of the sample. Then the valve in between the burette and the base plate was closed. Before placing the soil in the mold a Cellulose Acetate sheet was placed vertically adjacent to the internal sides of the polystyrene to keep the soil moisture from being absorbed by the polystyrene. Then a thin layer of antifreeze was applied on the sheet to minimize friction between the soil sample and the sheet during heaving. A desired percentage of water was mixed with the soil in an electrical mixer for 15 minutes and a

small sample from this mix was taken to determine the exact water content in the soil. Then the soil was placed in the apparatus in three equal layers. A thermistor was inserted before the placement of a new layer. Each layer was compacted with 25 evenly distributed blows of a hammer (76 mm diameter and 47 kN weight) using a 300 mm free drop. After compacting the third layer, the surface of the soil was trimmed to a distance of about 20 mm below the top of the polystyrene, thus the volume of the sample could be calculated.

To determine the weight of the soil used, the whole apparatus was weighed before and after placing the soil. Herein, knowing the volume, the water content, and the weight of soil used, the dry unit weight, the void ratio, and the degree of saturation were calculated. The valve was then opened and the sample was left for 24 hours under a 120 mm water head above the top of the sample from the burette to ensure a high degree of saturation. To find the exact degree of saturation, the apparatus was weighed again at the end of the soaking period, and thus the weight of water entered during soaking was determined. The sample was then ready for the test.

4.3.5.2 Samples with Cement

The procedure for the preparation of this sample was the same as above except the following:

1. The water was mixed with the desired percentage of cement to form a paste. Then this paste was mixed with the soil for 15 minutes. To decrease the time of setting to 72 hours, High- strength cement was used for all samples.
2. After the surface of the soil was trimmed, the sample was covered with plastic for 72 hours for hydration. After 48 hours the valve was then opened and the sample was left for 24 hours under a 120 mm water head above the top of the sample from the burette to ensure a high degree of saturation.

4.3.6 Test Procedures

The test procedures for all the samples were typical and could be summarized as follows: (1) dead weight was placed on the top of the sample, (2) the water level in the burette was lowered to mid-depth of the sample and the valve was kept open throughout the freezing process Fig.4.7., (3) the initial sample height was measured and the thermistor readings were recorded, (4) the apparatus was moved to the freezer and the heating cables were connected to the electrical supply, (5) the burette was covered with a perforated plastic cover to prevent water evaporation during freezing, (6) during the freezing process, readings of temperature, sample height, water level in the burette were recorded every four hours, (7) when the freezing front reached the bottom of the sample (i.e., no more water could be sucked up) the apparatus was taken out of the freezer, (8) just after taking the apparatus out of the freezer, it was weighted again to find out the exact amount of water sucked during freezing, (9) after thawing, the soil was taken out of the mold and water content was determined again.

4.3.7 Test Results

For each of the nine tests, a curve was plotted to show the relationship between the time and the heave as given in Figures 4.8, 4.9., and 4.10. Also Table 4.3. summarizes the amount of water gained during freezing and the heave of the nine soils.

Soil C shows that 3% cement reduces the heave from 2.7 to 0.5 mm, i.e., more than 80% reduction and the treated material still functions as a soil not as a hardened material, which is very important for soil-steel structures. Also soil B needed 3% to eliminate the heave, and still functions as a soil material. Furthermore, Table 4.3. shows a decrease in the amount of water sucked up during freezing, with the increase of cement content. Thus, the cement does not only reduce the heave but also protects the soil from the loss of bearing capacity during spring time, which is extremely important for soil-steel structures

Comparing the results of the three soils, one can conclude that there is a direct relation between the amount of fine materials in the soil and the amount of cement required to stop or reduce frost heave considerably. For example, 3% cement was not enough to stop frost heaving or even to reduce it considerably. 5% cement reduced the heave from 6.5 mm to 2.2 mm. Therefore, for some soils, the amount of cement required to reduce frost heaving considerably could be greater than that required to keep the soil as a semi-hardened material. Therefore, further research is required to classify the soils adequate for this treatment and to determine the corresponding percentage of cement.

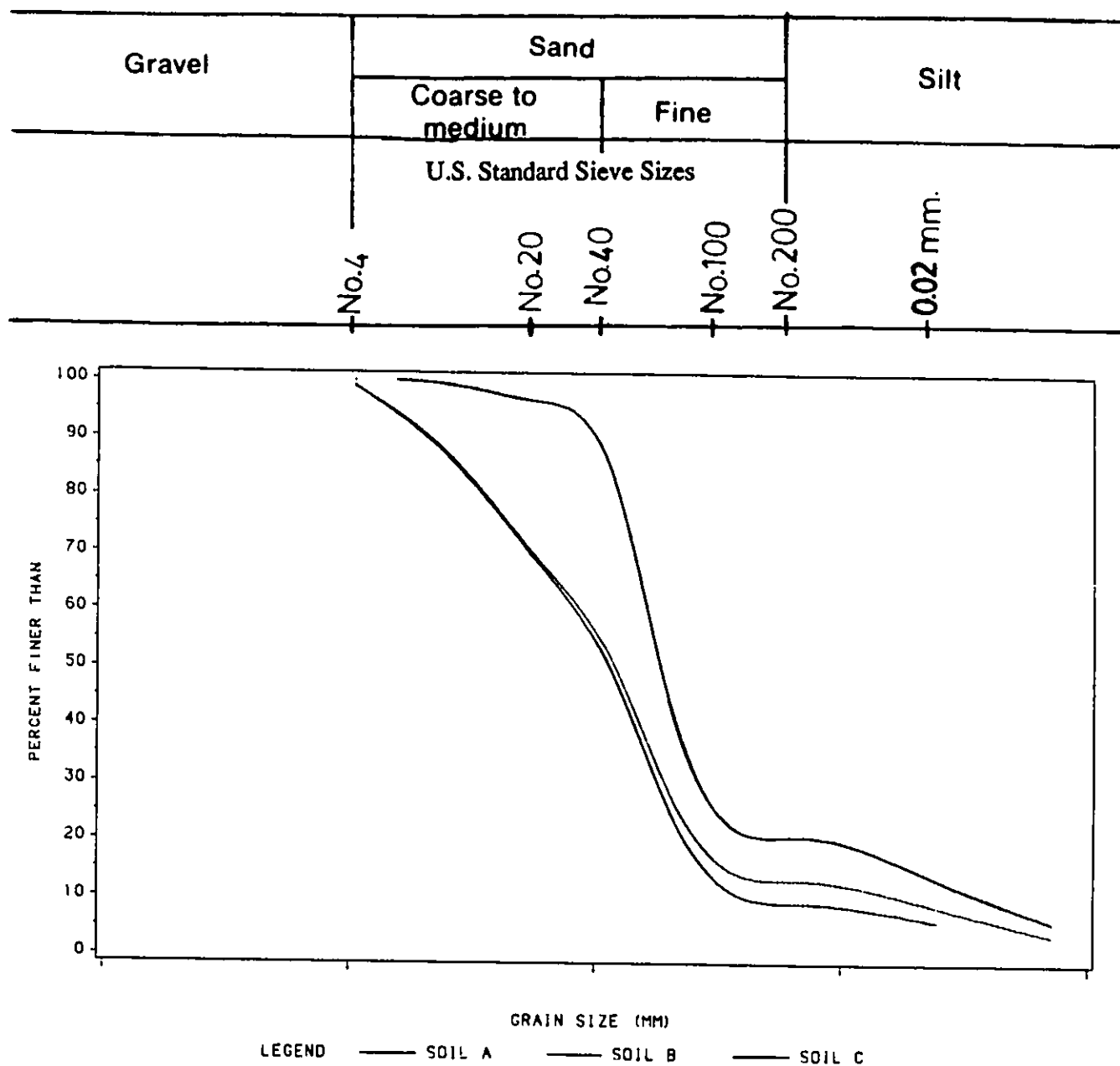


Figure 4.1: Grain size distribution of soils A, B, and C without cement.

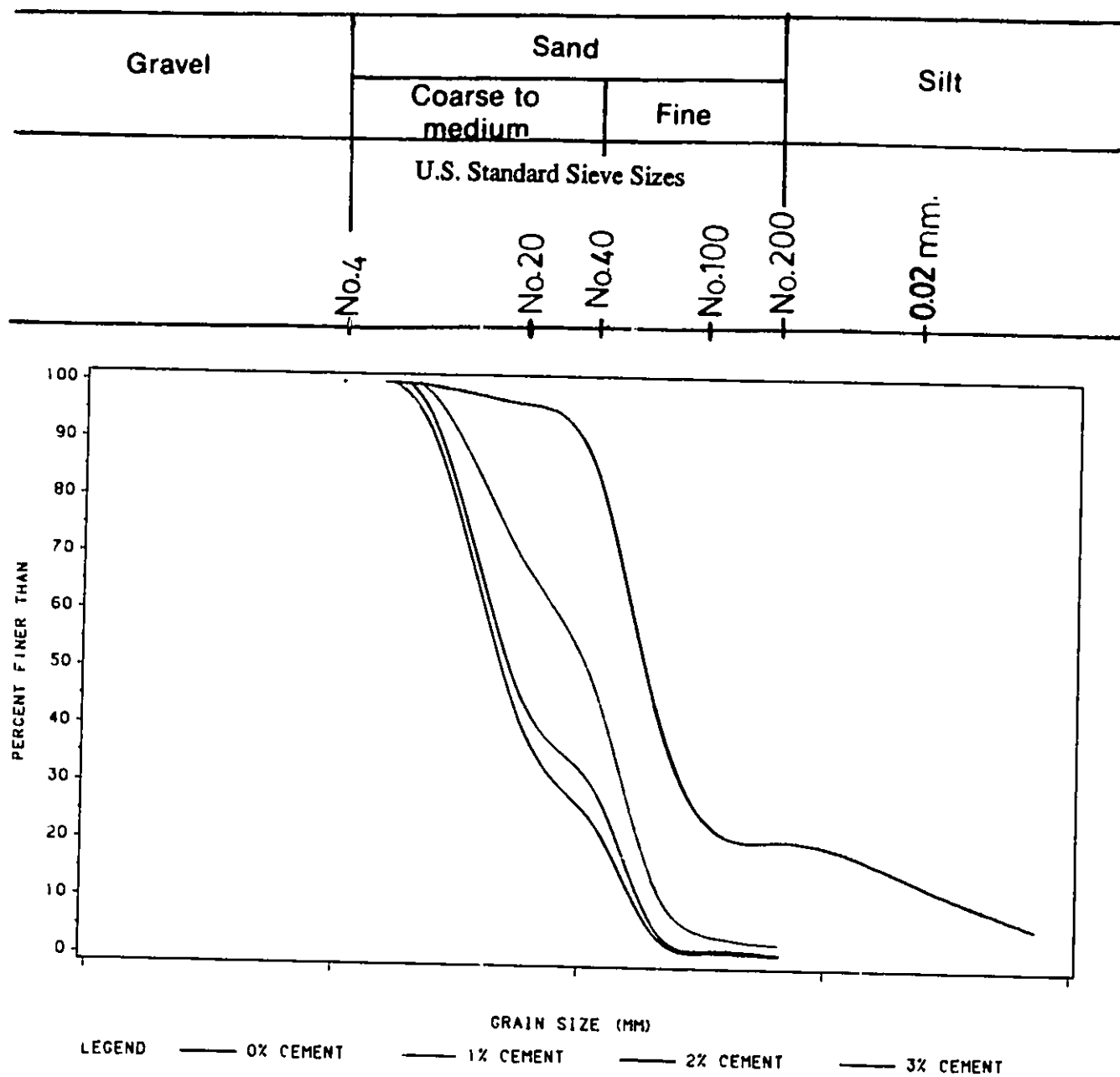


Figure 4.2: Grain size distribution of soil A with cement.

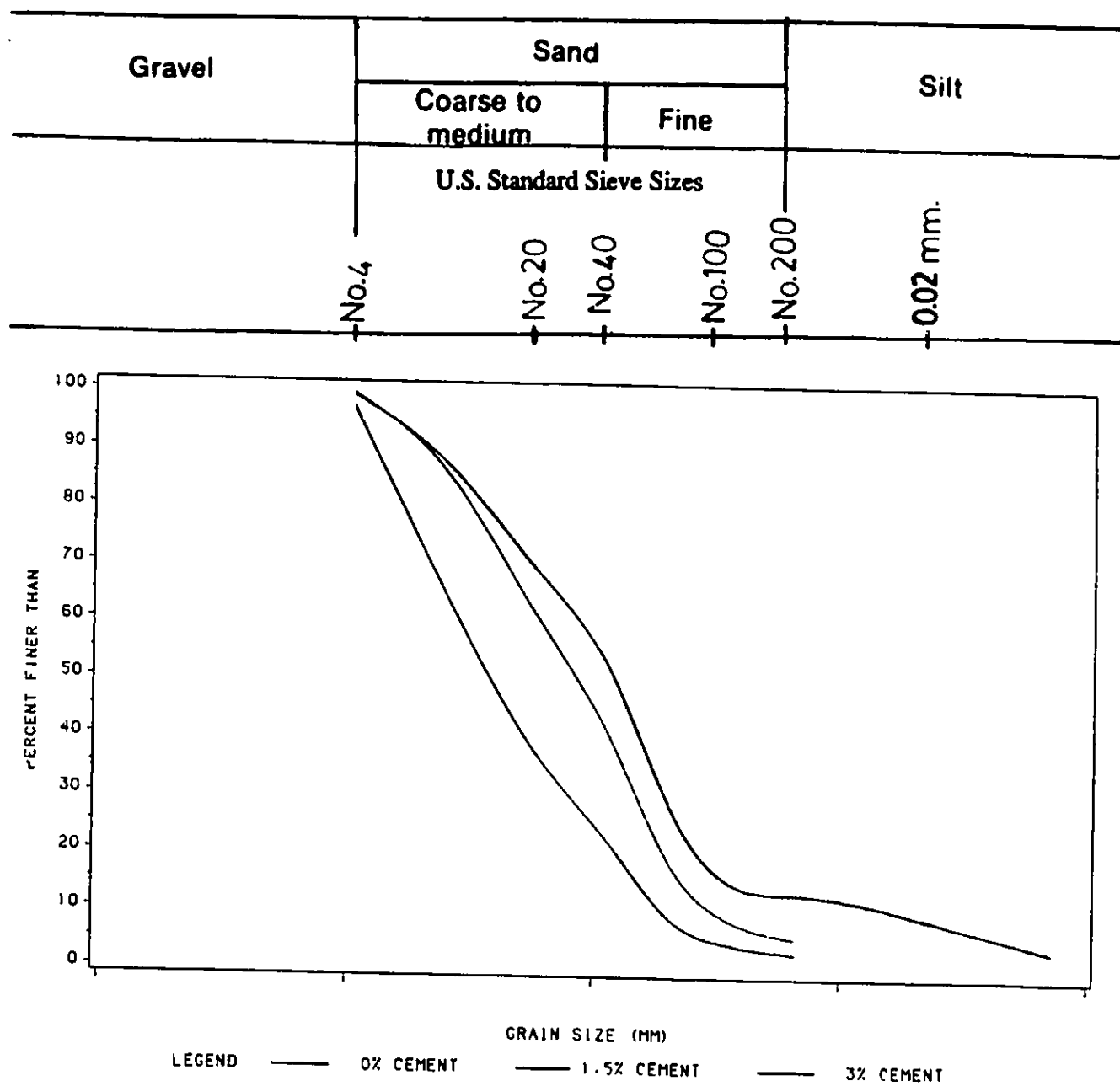


Figure 4.3: Grain size distribution of soil B with cement.

Gravel	Sand		Silt
	Coarse to medium	Fine	
U.S. Standard Sieve Sizes			
	No. 4	No. 20 No. 40	No. 100 No. 200
			0.02 mm.

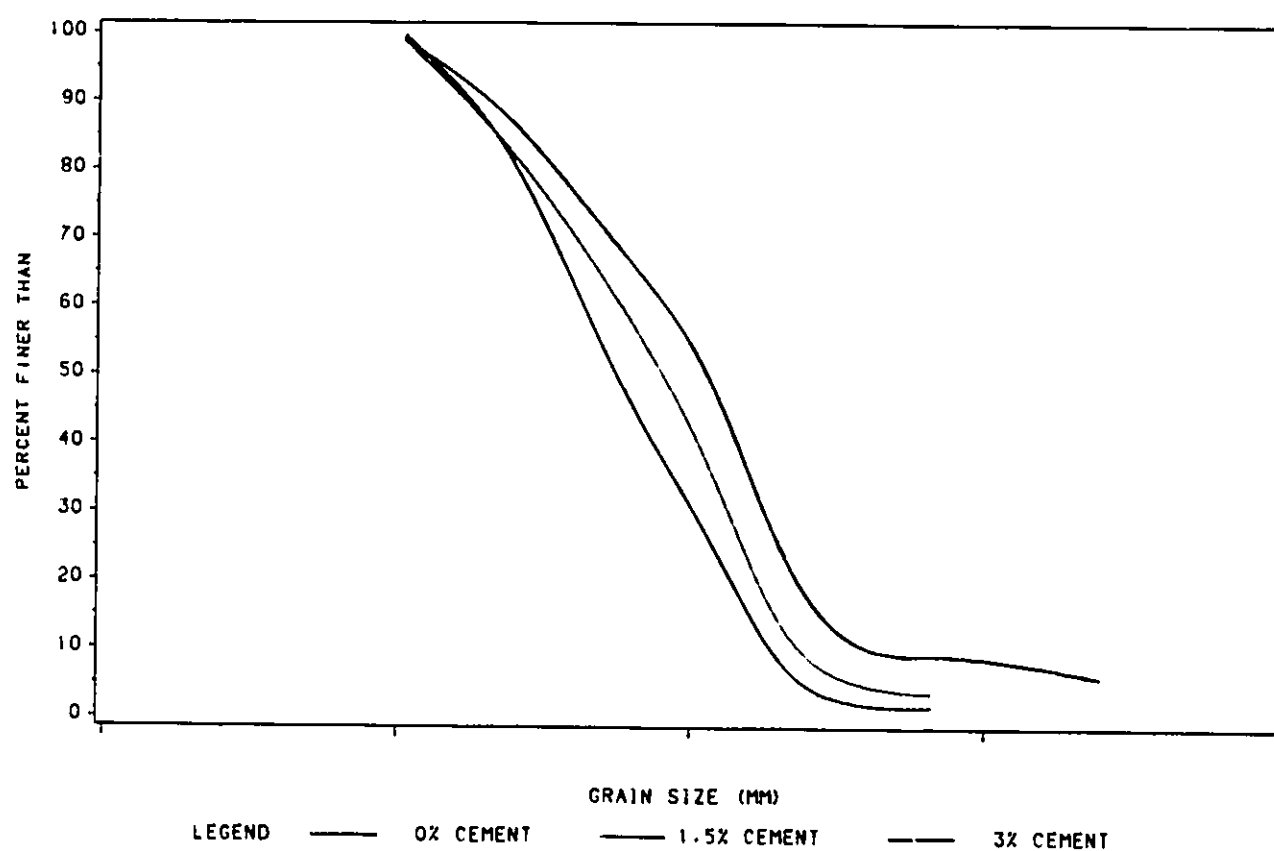


Figure 4.4: Grain size distribution of soil C with cement.

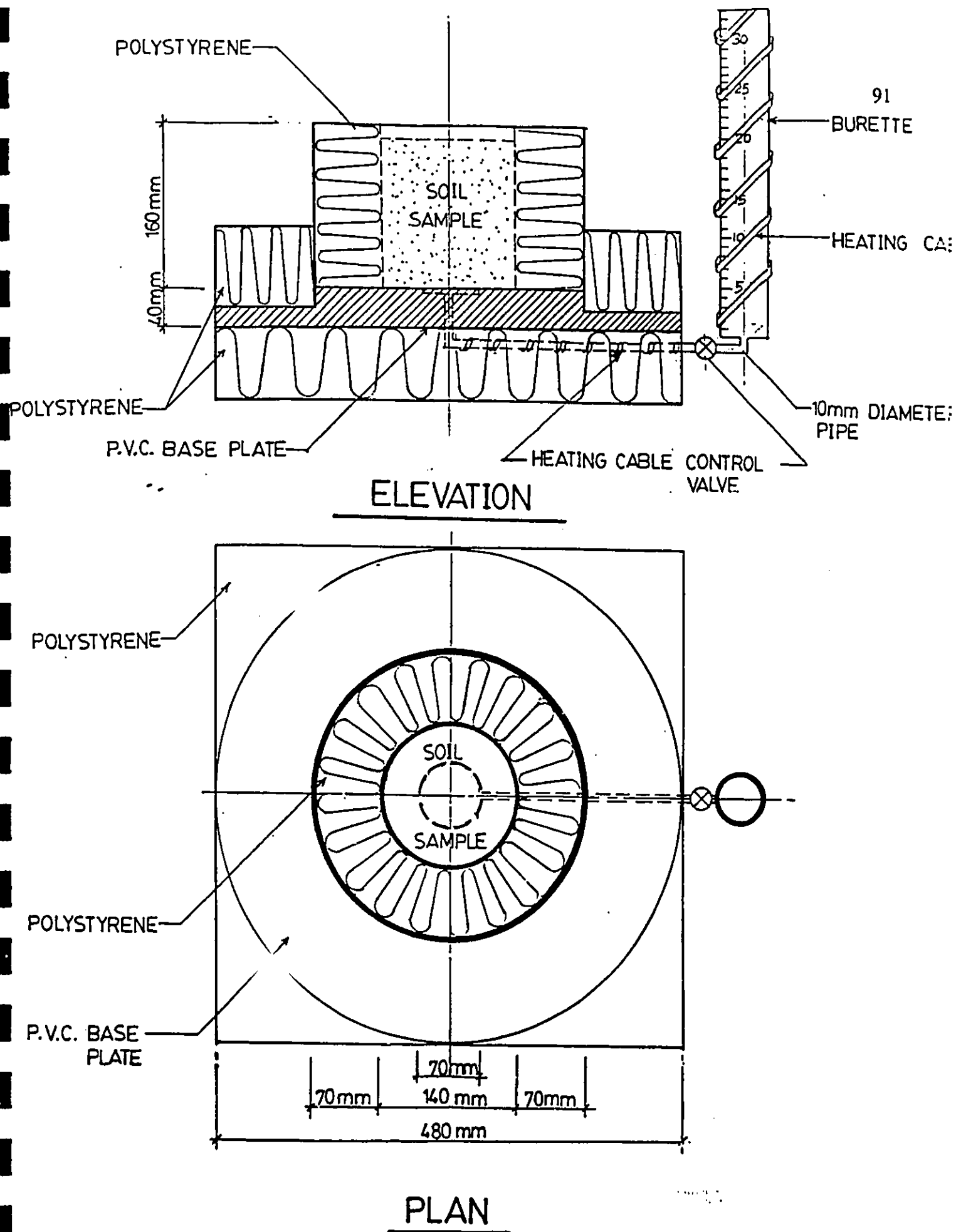


Figure 4.5: Detailed drawing of the frost cell.

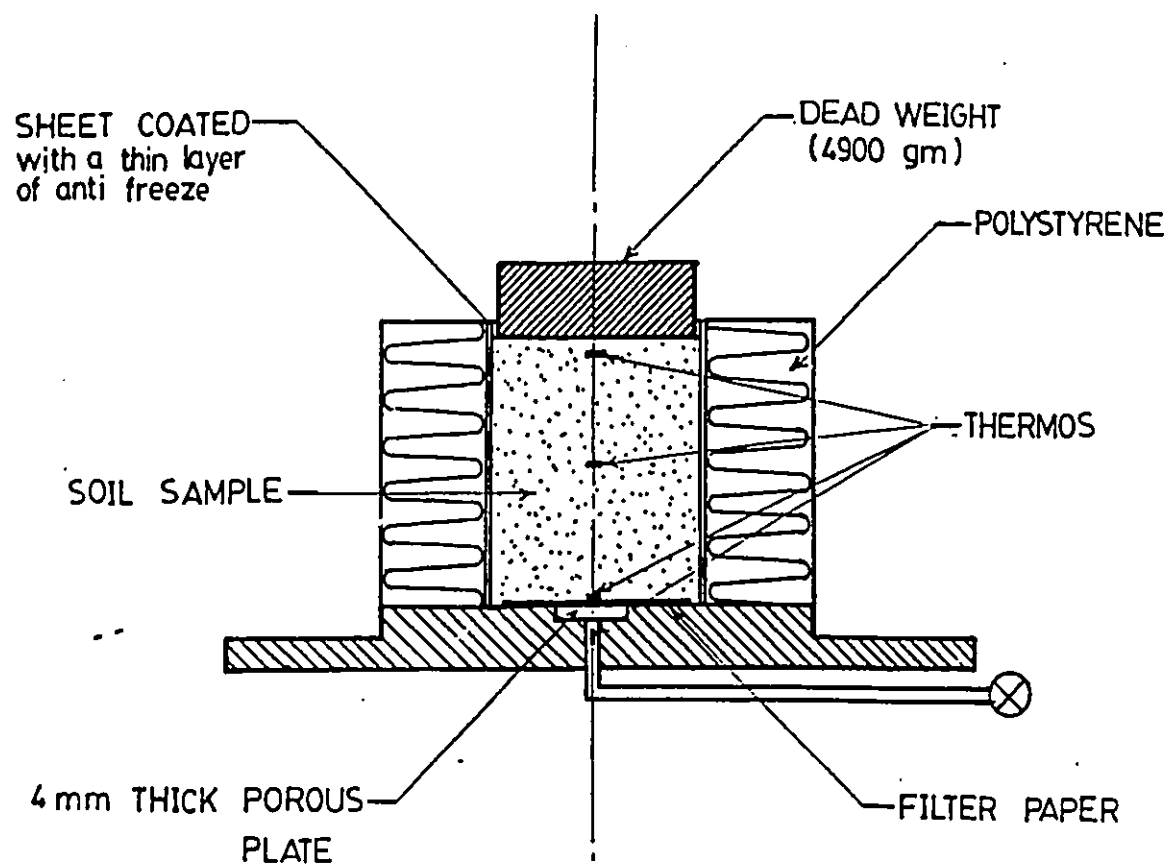


Figure 4.6: Cross-section of the frost cell.

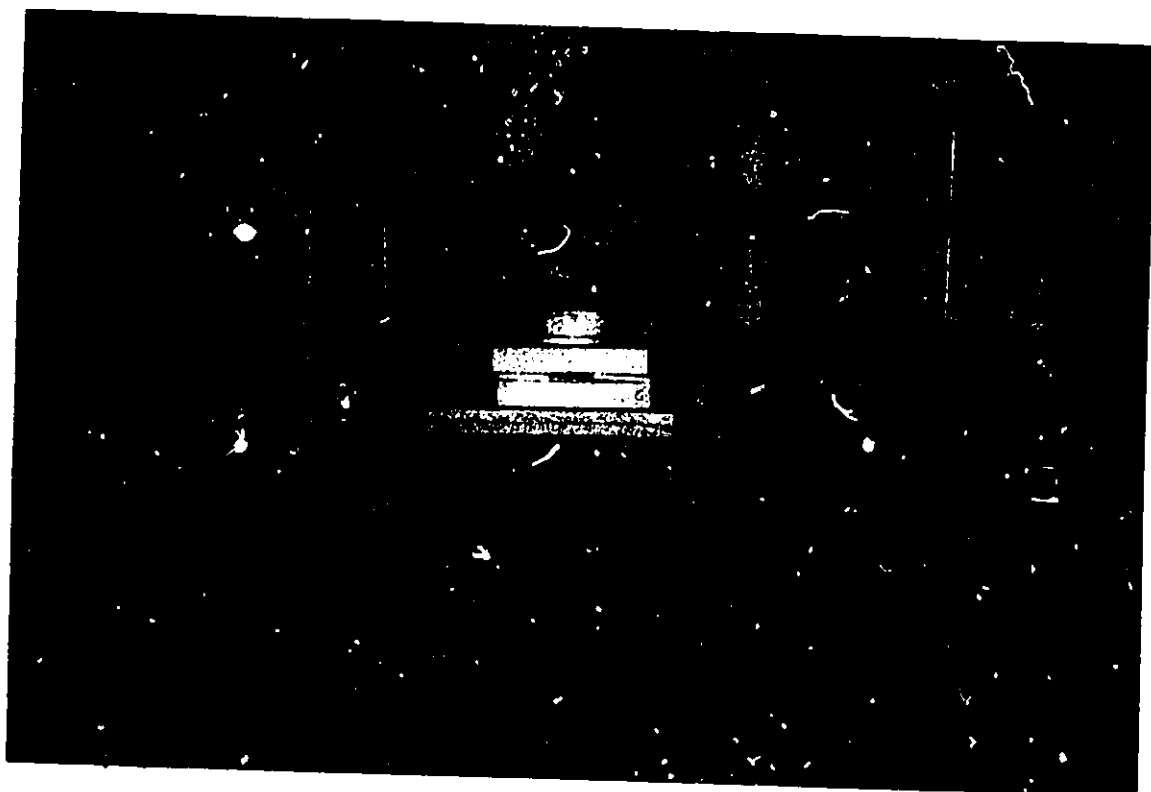
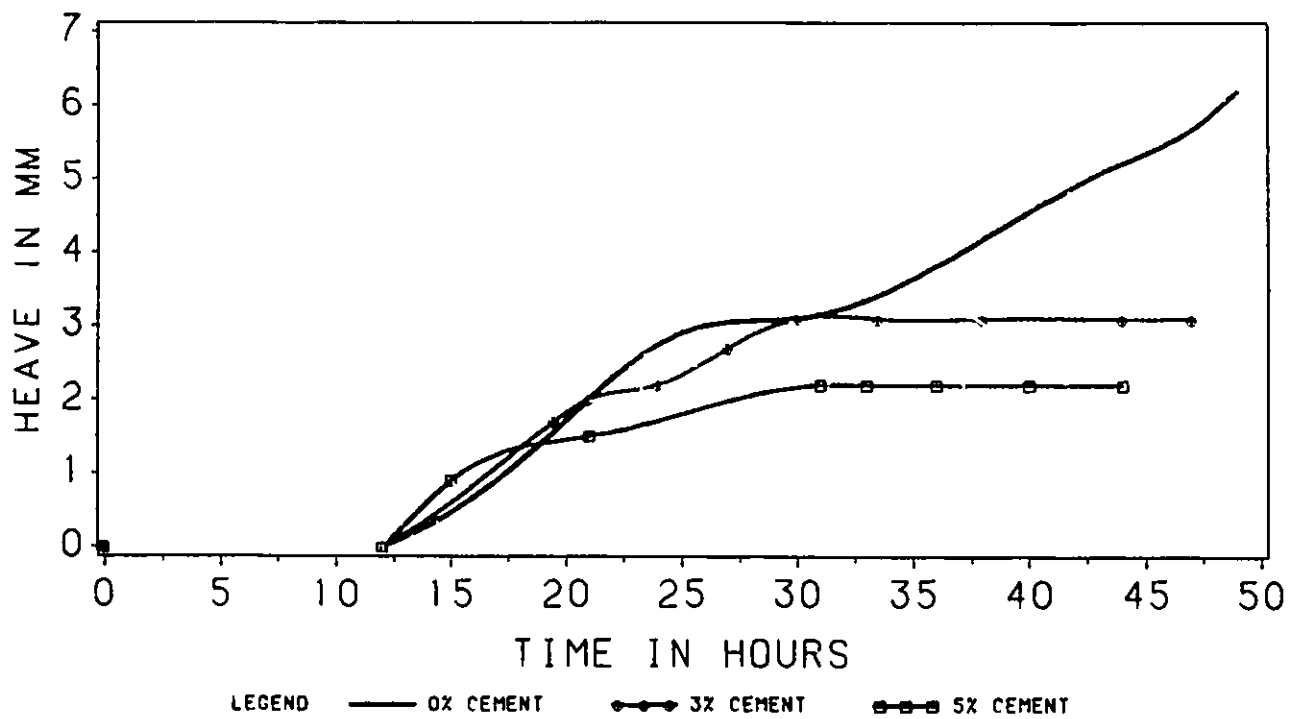


Figure 4.7: The frost cell inside the freezer.

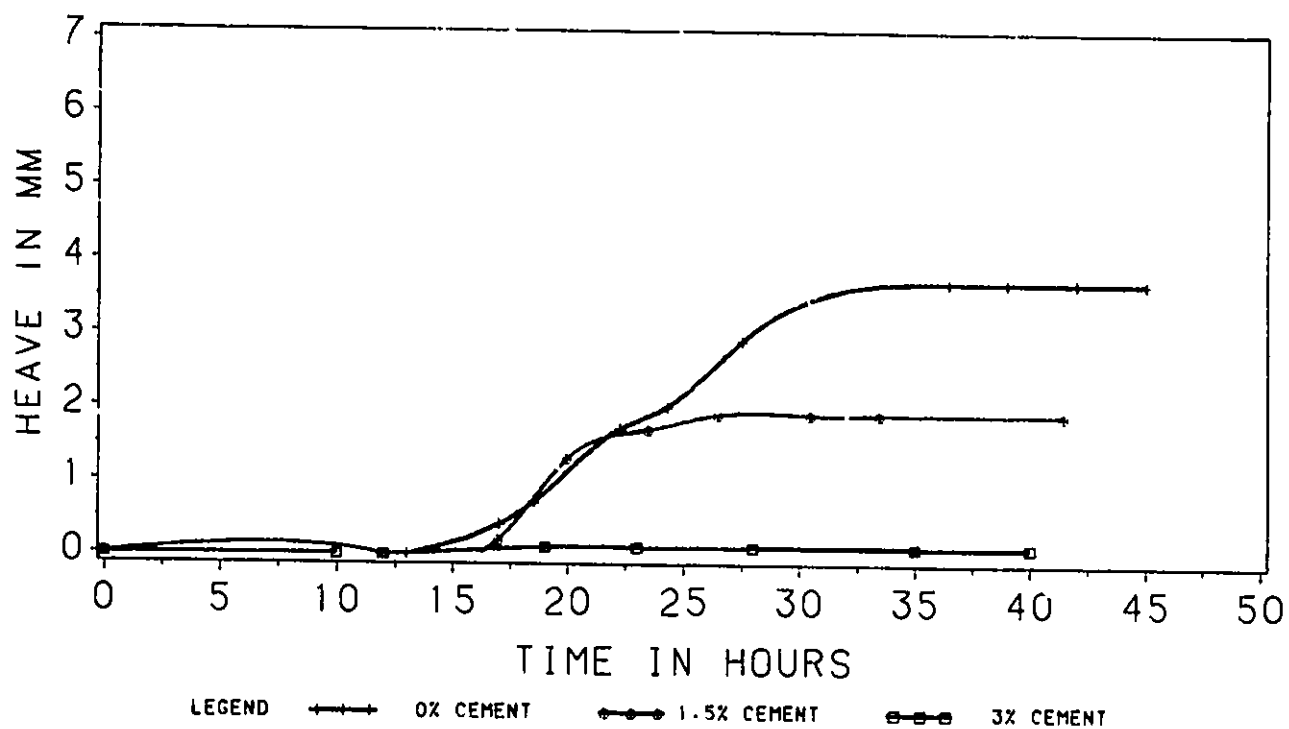
**FROST ACTION TEST
ON SOIL A
80% SAND AND 20% SILT**



**OPEN SYSTEM
SURCHARGE PRESSURE 3.4 kPa**

Figure 4.8: Experimental results of the heave for different percentages of cement with soil A.

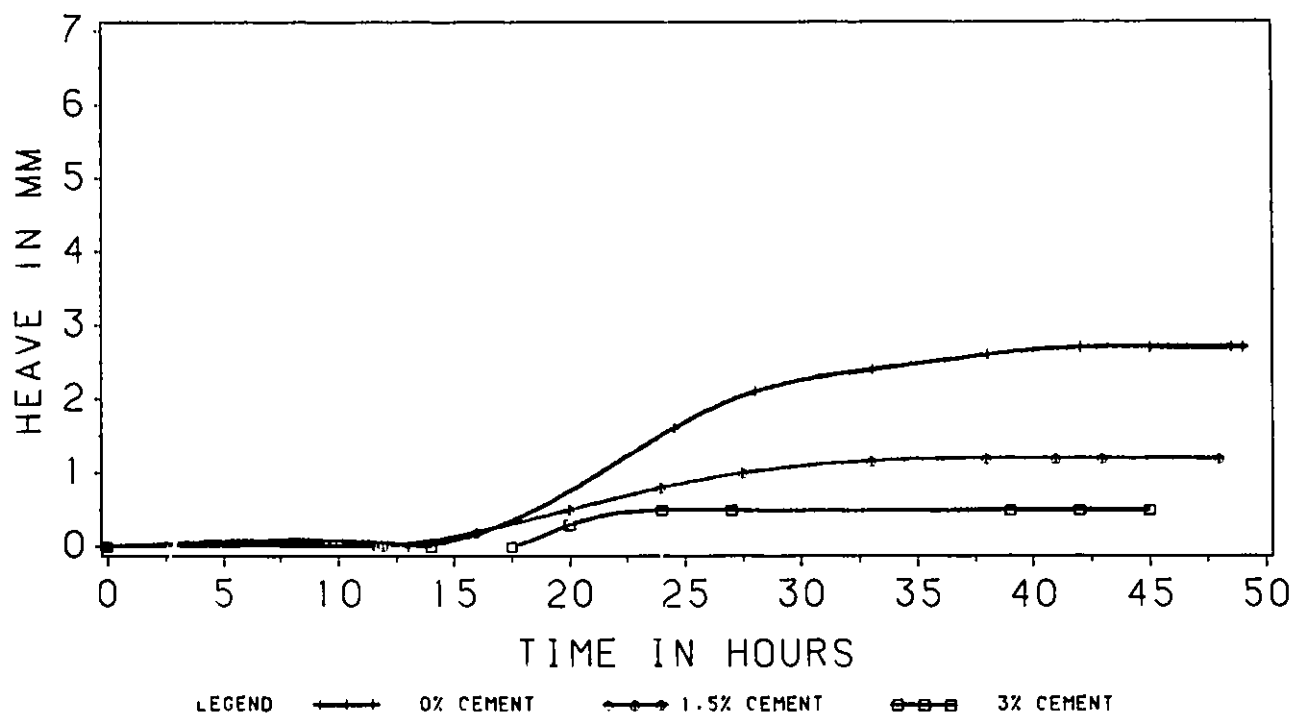
**FROST ACTION TEST
ON SOIL B
87% SAND AND 13% SILT**



**OPEN SYSTEM
SURCHARGE PRESSURE 3.4 kPa**

Figure 4.9: The experimental results of the heave for different percentages of cement with soil B.

**FROST ACTION TEST
ON SOIL C
91% SAND AND 9% SILT**



**OPEN SYSTEM
SURCHARGE PRESSURE 3.4 kPa**

Figure 4.10: The experimental results of the heave for different percentages of cement with soil C.

SOIL TYPE	CBR
Raw Soil	43
2% cement by weight, age 7 days	255
2% cement by weight after 60 cycles of freeze-thaw	258
4% cement by weight, age 7 days	485
4% cement by weight after 60 cycles of freeze-thaw	574

Table 4.1: Permanency of bearing value of cement-modified granular soil

Soil Title	% of Silt	% finer than 0.02mm	c_u	c_z	Soil Classification		G_s
					AASHTO System	Unified System	
A	20	13	19.9	7.30	A-2-4	SP-SM	2.62
B	13	8	24.8	5.08	A-1-b	SP-SM	2.65
C	9	6	7.73	1.73	A-1-b	SW-SM	2.67

Table 4.2: Geotechnical properties of soils A,B, and C.

SOIL		Dry Density (g/cm)	Before Freezing		Water Gained During Freezing (g)	Frost Heave (mm)
Sample	Cement %		Water Content %	Degree of Saturation %		
A	0	1.87	14.7	95	93.5	6.2
	3	1.84	14.0	90	70.0	3.1
	5	1.85	15.6	93	47.0	2.2
B	0	2.03	10.6	92	72.0	3.7
	1.5	2.05	10.5	95	33.0	1.9
	3	2.08	9.7	95	5.0	0.1
C	0	2.00	10.71	85.0	85	2.7
	1.5	2.08	10.20	96.0	65	1.2
	3	2.01	12.3	93.8	23	0.5

Table 4.3: Summary of the frost heaving test results on soils A, B, and C.

Chapter V

EVALUATION AND RECOMMENDATION

Because of the frequency of the distress in soil-steel structures, it behooves us to put together all the relevant pieces of theoretical and experimental information to provide a final evaluations and recommendations of the present codes in terms of frost heaving. In the design of structures constructed on soil subjected to cycles of freezing and thawing, beside the dead and live loads, the loads caused by heaving pressure must be taken into consideration unless the conditions allowing frost heaving are prevented. Since, heaving forces are very difficult to predict because they are related to the degree of restraint imposed during soil freezing, and since it is very difficult to estimate the loss of bearing capacity during thawing, the logical solution is to prevent the occurrence of frost action in the soil supporting these structures. Hence, the experience in building durable roads and highways is directed herein to achieve the greatest possible strength of the soil around the soil-steel structure.

5.1 The Structure of Backfill Material

According to the present codes of the American Iron And Steel Institute, the structure backfill material should conform to one of the soil classification from AASHTO Specification M-145 as follow "for heights of fill less than 3400 mm, A-1, A-3 , A-2-4, and A-2-5. And for heights of fill of 3400 mm or more, A-1 and A-3 may be satisfactory". The codes do not specify a certain limit for the percentage of the fine

material. For example soil A-1 could have up to 25% silt and/or clay, also, soils A-2-4 and A-2-5 could have up to 35%, which make them very frost-susceptible and prone to the loss of bearing capacity especially with the high degree of compaction usually performed around the culvert.

Moreover, the present code of Ontario specifies three soil groups according to the grain size (coarse, medium and fine) without limiting the percentage of the fine materials. For example, most of the soils in group II have at least 12% silt and/or clay. If any of these soils were well graded, the percentage of the fine materials should not exceed 3% to avoid frost action problems. The present code does not specify any group for the soils containing 5% to 12% passing sieve #200. Also, soils in group III have a very high percentage of fine materials, which makes them very frost susceptible soils.

Furthermore, another theory about the loss of soil support around the culvert is that there is a continuing loss of backfill material (fine sands and silts) every time the water rises and falls in the structure. In another words, even if the soil around the culvert were not frost susceptible one, the existence of high percentage of fine material may make the soil lose its support every time the water in the structure falls below the water level in the backfill.

Since the fine materials are that detrimental, their percentage must be specified in the codes. Casagrande's criterion is believed to be the most reliable one; it is followed by most of the provinc highway departments. Although the U.S. Corps of Engineers has done extensive research on frost action, Casagrande's criterion is still followed in the design of roads and airfield pavements. Similarly, this criterion must be considered in the design of the soil-steel structures. If the available soil in the site has a higher percentage of fine material than that specified by the criterion, this soil must be rejected and treated

or replaced by better quality soil unless it is proven experimentally that this soil is not a frost susceptible soil.

Moreover, there must be continuous inspection of backfilling operations, especially for the backfill subjected to passive earth pressure.

5.2 Soil Envelope Dimension

The present code of Ontario requires that the backfill extend transversely at least half the span on each side beyond the springing lines for the conduit. The American Iron And Steel Institute requires at least one diameter away from the conduit surface at mid-height. Both codes do not specify any thickness for the bedding or the backfill underneath the conduit. Hence, these dimensions are specified regardless of freezing penetration depth at each area. Therefore, if the frost penetration depth is greater than the specified thickness of the backfill, the regular soil which could be silty soil (frost susceptible) will freeze forming ice lenses and high pressure on the conduit and may lose its supporting capacity during thawing. To avoid such troubles, the backfill material must extend at least beyond the frost penetration depth. Also, the bedding soil, underneath the bottom plate must be non frost susceptible soil and must extend to at least the same depth. The frost penetration depth can be determined using the design curve and the freezing index at that particular location as explained in 2.1. Site supervision must be provided to prevent any frost susceptible material to lay within the frost penetration depth, especially where the soil is subjected to passive earth pressure.

5.3 Soil Material of The Embankment

The present codes do not specify any soil group for the part of the embankment above the culvert as a protection against the change in the backfill structure. The embankment material may affect the backfill material around the culvert. For example, the backfill around one of the culverts in Ontario [77] had a low percentage of fine material which made it non-frost susceptible soil, but the embankment material above that culvert had about 57% by weight clay and silt. It is very possible that these fine materials segregate on the backfill around the culvert forming a thin layers of fine material and changing the backfill to frost-susceptible material.

To ensure the structure of the backfill remains the same through the years of operation, the following must be considered:

1. The embankment material above the culvert must be selected according to the same frost criterion as the backfill material around the culvert.
2. Head walls must be provided at both ends of culvert. However, the principal purpose of the end walls on steel pipe culverts is hydraulic efficiency to prevent scour at the inlet, undermining the outlet, and to increase the capacity. They also protect the backfill soil from the fine materials in the stream. These fine materials may deposit on the fill soil around the culvert increasing the frost susceptibility of the soil.

5.4 Construction Technique

The present codes do not specify a certain compaction or construction technique to avoid the accumulation of fine materials in layers within the selected backfill. However, a thin layer of silt or clay could be accumulated during dumping the backfill which could

be very detrimental. Beskow (1935) [7] stated "In sands, if an ever so thin layer of fine material, silt, fine silt, or clay seam exists, an appreciable ice layer can form under favorable circumstances.". While the sand may appear at the surface to be non-frost heaving soil, the existence of a thin layer of fine silt may make it strongly frost-heaving soil, as shown in Fig. 5.1.a and 5.1.b. Hoekstra (1969) [24] measured the heaving pressure of a stratified sample of sand and silt. He found that when the frost line moved from sand to silt the rate of pressure increased sharply; when the frost line moved from silt to sand the pressure levelled off. This could occur even if the backfill material had a very small content of fine material. Smith (1946) [28] recommended placing the gravel in layers of not more than 25 mm thickness to avoid segregation of material.

5.5 CONCLUSIONS

1. Frost action is one of the reasons of distress in soil-steel structures
2. The present codes of soil-steel structures completely overlooked the effect of frost action
3. In the design of soil-steel structures subjected to cycles of freezing and thawing, beside the dead and live loads, heaving pressures must be considered
4. Cement-modified soil could be used with soil-steel structures as a remedy for the frost action.

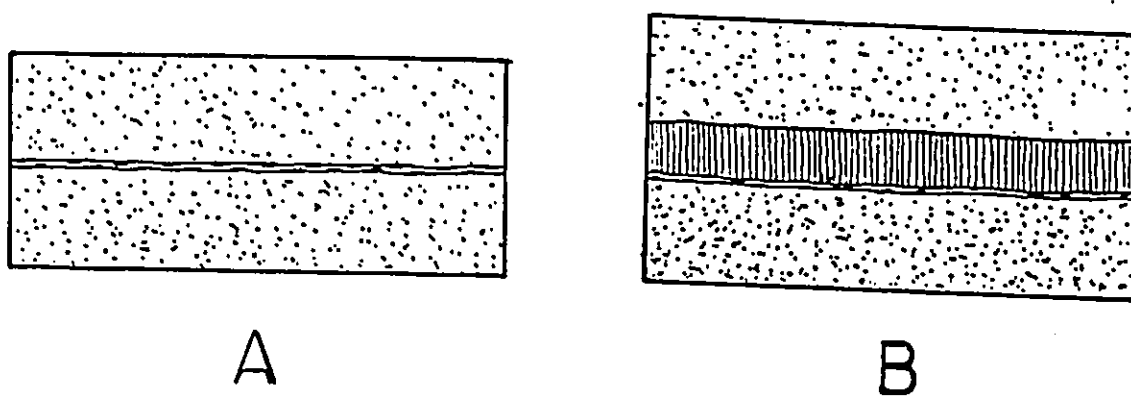


Figure 5.1: (A) Fine silt or clay strata in an otherwise non-frost susceptible soil (sand) before freezing. (B) Thick ice layer at the boundaries between soil layers.

Appendix A

PIPE ARCH EXPERIMENT MODEL

A.1 Summary of soil properties

Specific Gravity	2.66
% Passing Sieve # 200	0.25
D ₁₀ . (mm)	0.22
Coefficient of uniformity C_u	1.99
Classification according to AASTHO System	A-3
Classification according to unified system:	SP

A.2 Mechanical properties of the culvert material

Utility Aluminum (3003-H14)	
Tensile Strength (MPa)	148.25
Yield Strength (MPa)	137.90
Modulus of elasticity (GPa)	70.00
Unit Cross Section Area (mm ² /mm)	0.87
Unit Moment of Inertia (mm ⁴ /mm)	3.70

Appendix B

EQUIPMENT SPECIFICATIONS

B.1 Pipe Heating Cable

Available from Heron Cable Industries Limited

440 Phillip St.

P.O.Box 940

Waterloo, ONT.

N2J 4C3

B.2 Thermistor Probe

Model ON-401-PP

Temperature range -20° to 100° C

available from OMEGA Engineering

P.O.Box 4047

Stamford, CT 06907-0047

REFERENCES

1. Abzhlimov, R.S. "Additional load developed as a result of freezing of backfill soil on walls of underground crossing", Plenn Publishing Corporation.
2. Aftab, A.M., Bakht, B., and Jaeger, G.L. (1989) "Mechanics of behavior of soil-steel structures" Annual Conference and the 9th Canadian Hydrotechnical Conference, Canadian Society For Civil Engineering
3. American Iron and Steel Institute (1984) "Handbook of steel drainage & highway construction products" Published by American Iron and Steel Institute, 1000 16th street, N W, Washington, D C 20036
4. Armstrong, M.D. and Csathy, T.I. (1963) "Frost design practice in Canada" Highway Res. Rec.33
5. Arvidson, W.D. and Morgenstern, N.R. (1974) "Water flow induced by soil freezing" Presented to the 27th Canadian Geotechnical Con. Edmonton, Alberta
6. Berg, R. and Johnson, T. (1983) "Revised procedure for pavement design under seasonal frost conditions" U.S. Army Corps of Engineers, Cold Regions Research & Engineering Laboratory, Special Report 83-27.
7. Beskow, G. (1935) "Soil freezing and frost heaving with special application to roads and railroads" Published By the Technological Institute Northwestern University Evanston, Illinois 1974
8. Brandt, G.H. (1973) "Chemical additive to reduce frost heave and water accumulation in soils" Highway Res. Rec.393

9. Burn K.N. (1976) "Frost action and foundations" National research Council Canada
10. Casagrande, A. (1931) "A New Theory of Frost Heaving" by A.C. Benkelman and F.R. Olmstead. Highway Research Board Proc. 11(1), p. 168-172.
11. Chapuis, R.P. (1988) "Two case history of major frost heaving in refrigerated building: thermal analyses, repairs, and prevention" Canadian Geotechnical Journal Vol.25
12. Comey, D. and Jacobs, J.C. (1967) "The frost susceptibility of soil and road materials" Road Research Laboratory Report LR90, Crowthorne, Berkshire, England.
13. Curtis, W.E. and Forbes, A.J. (1963) "Determination of cement content of soil-cement mixtures" Highway Res.Rec.36
14. Dow Chemical Company (1988) "Soil insulation" and "Highway insulation" Dow Chemical Company, Toronto, Ontario.
15. Edil, B.T. and Bahmanyar, H.G. (1983) "Stresses in buried conduits caused by freezing front" Elsevier Science Publishers B.V.
16. Everett D.H. and Haynes, J.M. (1965) "Capillary properties of some model pore system with special to frost damage" RILEM Bull. n.s. 27, P. 31-38
17. Farouki, O.T. (1986) "Thermal properties of soils" Trans Tech Publications
18. George, G.P. and Davidson, D.T. (1963) "Development of freeze-thaw test for design of soil-cement" Highway Res. Rec.36
19. Gold, I.W. (1957) "A possible force mechanism associated with the freezing of water and porous material" Highway Res. Board, Bull. 168

20. Haley, J.F. and Kapler, C.W. (1952) "Cold Room Studies of frost action in soil" Highway Res. Board Special Report No.2 pp.246-267
21. Haley, J.F. and Kapler, C.W. (1953) "Cold Room Studies of frost action in soil, a progress report" Highway Res. Bull.71
22. Hafez, H.H. (1982) "Soil-Steel Structures Under Shallow Cover" PH.D. Dissertation University of Windsor
23. Hoekstra, P., Chamberlain, E., and Frate, T. (1965) "Frost-heaving Pressures" Highway Res. Rec.101
24. Hoekstra, P. (1969) "Water movement and freezing pressures" Soil Sci. Soc. Amer. Proc., Vol.33
25. HRB Special report No.2 "Frost action on soil" Symposium Presented at the thirteenth annual meeting January 1951
26. Jessberger, H.L. and Carbee, D.L. (1968) "Influence of frost action on the bearing capacity of soils" Highway res. Rec.304
27. Jessberger, H.L. (1979) "Ground freezing" Elsevier Scientific Publishing Company.
28. Johnson, A.W. (1952) "Frost action in roads and airfields" Highway Res. Board Special Report No.1
29. Joseph E. Bowles (1978) "Engineering properties of soils and their measurement" McGraw Hill Book Company.
30. Jumikis, A.R. (1956) "The soil freezing experiment" Highway Res. Board Bull.135
31. Jumikis, A.R. (1966) "Thermal soil mechanics" Rutgers University Press, New Brunswick, New Jersey

32. Jumikis, A.R. (1967) "Introduction to soil mechanics" Rutgers University Press, New Brunswick, New Jersey
33. Jumikis, A.R. (1973) "The soil-culvert-temperature system upon freezing" Organization for Economic Co-operation and Development
34. Jumikis, A.R. (1977) "Thermal geotechnics" Rutgers University Press, New Brunswick, New Jersey
35. Kawala, E.L. (1966) "Cement-treated subbase practice in U.S. and Canada" Journal of the Highway Division Proceedings of the American Society of Civil Engineers No.4947
36. Keene, P. (1951) "Remedies and treatment by Connecticut highway department" Highway Res. Board Special Report No.2
37. Kenneth, A.L. (1954) "Frost design criteria for pavements" Highway Res. Board Bull.71
38. Laba, J.T. (1970) "Lateral thrust in frozen granular soils caused by temperature change" Highway Res. Rec. No.304
39. Laba, J.T. and Aziz, K.A. (1972) "Pressure-time relationship in laterally stressed frozen granular soils" Highway Res. Rec. No.393
40. Lambe, T. W. (1951) "Soil testing for engineers" New York. John Wiley & Sons, Inc. London . Sydney.
41. Lambe, T.W. (1956) "Modification of frost-heaving of soils with additives" Highway Res. Board Bull.135
42. Luong, M.P. (1988) "Collapsing Peak up of a large highway steel pipe-arch" Proceedings: 2nd Int Con. on Case Histories in Geotechnical Eng., June 1-5, 1988, St. Louis, Mo., Paper No.6.88

43. McRoberts, E.C. and Morgenstem, N.R. (1974) "Pore water expulsion during freezing" Canadian Geotechnical Journal Vol.12
44. McRoberts, E.C. and Nixon, J.F. (1975) "Some geotechnical observation on the role of surcharge pressure in soil freezing" Proc., Conf. on Soil Water Problems in Cold Regions, Calgary
45. McRostie, G.C. and Schriever, W.R. (1967) "Frost pressures in the tie-back system at the National Arts Center excavation" Engineering Journal, March 1967
46. Mirza, C. and Porter, W.A. (1981) "Construction considerations and controls for soil-steel bridge structures" Canadian Journal of Civil Eng. Vol.8
47. Moore, R.G. (1986) "Observed signs of distress in soil steel structure"
48. Morgenstem, N.R. and Sego, D.C. (1980) "Performance of temporary tie-backs under winter conditions" Canadian Geotechnical Journal Vol.18
49. Norling, L.T. (1963) "Standard laboratory tests for soil-cement-development, purpose and history of use" Highway Res.Rec.36
50. Nussbaum, P.J. and Canadian, L. (1975) "Load factors for concrete pipes bedded in cement-treated soil" Portland Cement Association
51. O.E.C.D. (1973) "Frost action on roads" Proceedings of the Symposium on frost action on roads held at the Norwegian Road Research laboratory in Oslo on 1st, 2nd and 3rd October 1973
52. Okada, K. (1987) "Study for icicle prevention by adiabatic treatment in railway tunnel" Quarterly Reports Vol. 28 no.1
53. Osler, J.C. (1967) "The influence of depth of frost penetration on frost susceptibility of soils" Canadian Geotechnical Journal Vol.4

54. Osler, J.C. (1970) "Some considerations of heave and heaving pressure in frozen soils" 6th. World Highway Conf. Montreal
55. PCA (1971) "Soil-Cement laboratory handbook" Portland Cement Association
56. PCA (1971) "Subgrade and subbases for concrete pavements" Portland Cement Association
57. PCA (1976) "Properties and uses of cement-modified soil" Portland Cement Association
58. PCA (1979) "Soil-Cement construction handbook" Portland Cement Association
59. Penner, E. (1957) "Soil moisture tension and ice segregation" Highway Res. Bull.168
60. Penner, E. (1958) "Pressure developed in a porous granular system as a result of ice segregation" Highway Res. Board Special Report 40
61. Penner, E. (1959) "The mechanism of frost heaving in soils" Highway Res. Bull.225
62. Penner, E. (1966) "Pressure developed during the unidirectional freezing of water-saturated porous materials" International Conf. on Low Temperature Science, Sapporo, Japan. Vol. 1, Part 2 p. 1401-1412
63. Penner, E. (1967) "Heaving pressure in soils during unidirectional freezing" Canadian Geotechnical Journal Vol.4
64. Penner, E. (1968) "Particle size as a basis for predicting frost action in soils" Soils and Foundations Vol. VIII.
65. Penner, E. (1969) "Frost heaving forces in Leda Clay" Canadian Geotechnical Journal Vol.7

66. Penner, E. (1971) "Heave and heaving pressure in frozen soils, Discussion"
Canadian Geotechnical Journal Vol.8
67. Penner, E. (1973) "Uplift forces on foundation in frost heaving soils" Canadian
Geotechnical Journal Vol.11
68. Penner, E. (1976) "Insulated road study" Transportation Res. Rec. 612
69. Penner, E. and Crawford C.B. (1983) "Frost action and foundation" National
Research Council of Canada
70. Penner, E. and Ueda, T. (1977) "The dependence of frost heaving on load
application" Proceedings, Symposium on Frost Action in Soils, University of
Lulea, Sweden Vol.I pp.92-101
71. Penner, E. and Ueda, T. (1978) "A soil frost-susceptibility test and a basis for
interpreting heaving rates" Proc. Third International Conf. on Permafrost, Vol. 1,
p.721-727
72. Penner, E. and Walton, T. (1978) "Effects of Temperature and pressure on frost
heaving' International Symposium on Ground Freezing, Ruhr Univ., Bochum,
Germany.
73. Robinsky, E.I. and Bessflug, K.E. (1973) "Design of insulated foundation"
Journal of the soil mechanics and foundation division, Sept. 1973
74. Sandegren, E. (1987) "Insulation against ice in railroad tunnels" Transportation
Res. Rec. 1150
75. Sayed, G.A., Bakht, B. and Jasger, L.G. (1990) "Soil-Steel Bridges Design And
Construction" to be published.
76. Scott, R.F. (1963) "Principles of soil mechanics" Addison-Wesley Publishing
Company, Inc.

77. Selig, E.T., Lockhart, C.W. and Lautensleger, R.W. (1979) "Measured Performance of Newtown Creek Culvert" Journal of the Geotechnical Engineering Division Sept. 1979
78. Smith, H.R. (1951) "Calcium chloride treatment of frost action" Highway Res. Board Special Report No.2
79. Sowers, G.B. and Sowers G.F. (1970) "Introductory soil mechanics and foundations" Macmillan Publishing Co. Inc. New York
80. Spangler, M.G. and Handy, R.L. (1982) "Soil engineering" Harper & Row, Publishers, New York
81. Sutherland, H.B. and Gaskin, P.N. (1973) "A comparison of the T.R.R.L. and C.R.R.E.L. tests for the frost susceptibility of soils" Canadian Geotechnical Journal Vol.10
82. Taber, (1930) "The Mechanics of Frost Heaving" Journal of Geology vol.38, pp.303-317
83. Turner, K.A. (1955) "Loss and recovery of bearing capacity of 30 New Jersey soil material as determined by field CBR tests 1954-5" Highway Res. Board bull.168
84. Turner, K.A. and Jumikis, A.R. (1955) "Loss of bearing capacity and vertical displacements of New Jersey Soils" Highway Res. Board Bull. 135
85. Vinson, T.S., Ahmadi, F., and Rieke, R. (1986) "Factors important to the development of frost heave susceptibility criteria for coarse-grained soils". Transportation Res. Rec.1089
86. Wang, M.C. and Roderick G.I. "Frost behavior of compacted soils" Highway Res. Rec.

87. Williams, P.J. (1966) "Pore pressures at a penetration frost line and their prediction" Norwegian Geotechnical Institute Pub.72
88. Williams, P.J. (1967) "Replacement of water by air in soil pores" Norwegian Geotechnical Institute Pub.72
89. Williams, P.J. (1968) "The nature of freezing soil and its field behavior" Norwegian Geotechnical Institute Pub.72
90. Winterkorn, H.F. and Fang, H.Y. (1975) "Foundation engineering handbook" Van Nostrand Reinhold Company
91. Yong, R.N. and Warkentin, B.P. (1966) "Introduction to soil behavior" The Macmillan Company, New York
92. Yong, R.N. and Osler, J.C. (1970) "Heave and heaving pressures in frozen soils" Canadian Geotechnical Journal Vol.8
93. Yong, R.N. and Janiga, P.V. (1971) "Field study of moisture movement and ground heave during freeze-up" Soil Mechanics Laboratory,
94. Yong, R.N. and Warkentin, B.P. (1975) "Soil properties and behavior" Elsevier Scientific Publishing Company.
95. Young, F. D. (1965) "Experimental Foamed Plastic base course" Highway Res. Rec.101.

VITA AUCTORIS

- 1962 Born on September 28th in Cairo, Egypt
- 1979 Graduated from Tawfikia High School, Cairo, Egypt
- 1984 Graduated from Ain Shams University, Cairo, Egypt,
with the degree of Bachelor of Applied Science in
Civil Engineering
- 1988 Accepted into the Faculty of Graduate Studies and
Research, University of Windsor, Windsor, Ontario,
Canada, in a programme leading to the degree of
Master of Applied Science in Civil Engineering